Appendix 12 – Groundwater Effects Report

WGA
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Te Awamutu Developments Limited

Harlow Lifestyle Village – Groundwater Effects Substantive Report

TECHNICAL ASSESSMENT

WGA260929

WGA260929-RP-HG-0010_D

October 2026



Revision History

REV	DATE	ISSUE	ORIGINATOR	CHECKER	APPROVER
A	18/08/2026	DRAFT	ECA	CHO	CHO
B	21/09/2026	FINAL	ECA	CHO	CHO
C	25/09/2026	FINAL	ECA	CHO	CHO
D	27/10/2026	FINAL	ECA	CHO	CHO

Authors

The qualifications and experience of the authors and technical director/reviewer of this report are summarised in the table below.

AUTHOR	QUALIFICATIONS	EXPERIENCE	
Clare Houlbrooke	BSc Earth Sciences, MSc (Hons, Tech) Earth Sciences	Hydrogeologist with over 25 years of experience. Clare focuses on sustainable management of groundwater resources and connected surface water systems. She gained broad experience in multiple hydrogeological disciplines during her 9-year stint as a Groundwater Scientist in two Regional Councils before becoming a consultant in 2011.	I confirm that I have read Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023 and referred to in the Fast-track Approvals Act 2024: Panel Convenors' Practice and Procedure Guidance. This assessment has been prepared in compliance with that Code.
Brett Sinclair	BSc Geology, MSc (Hons) Geology, Postgraduate Certificate in Hydrogeology and Engineering Geology of Tropical and Subtropical Regions	Brett is a Senior Principal Hydrogeologist with more than 30 years' experience in hydrogeology, geology, water management, water quality assessment and environmental effects mitigation.	I confirm that I have read Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023 and referred to in the Fast-track Approvals Act 2024: Panel Convenors' Practice and Procedure Guidance. This assessment has been prepared in compliance with that Code.
Toby Beisly	BSc (Tech), Environmental Sciences and Geology, MSc (Research), Environmental Sciences	Toby is a Graduate Hydrogeologist. Toby has been involved in a wide range of projects across the Waikato, ranging from hydrogeological drilling and data analysis, hydraulic testing and analytical modelling.	I confirm that I have read Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023 and referred to in the Fast-track Approvals Act 2024: Panel Convenors' Practice and Procedure Guidance. This assessment has been prepared in compliance with that Code.
Erica Cammack	BSc (Hons) Environmental Science	Erica is a Hydrogeologist and Engineering Geologist with over 10 years of experience. She has experience in hydrogeology, engineering geology and geotechnical engineering from a broad range of projects throughout NZ and Australia.	I confirm that I have read Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023 and referred to in the Fast-track Approvals Act 2024: Panel Convenors' Practice and Procedure Guidance. This assessment has been prepared in compliance with that Code.

CONTENTS

1 INTRODUCTION 1
1.1 Background 1
1.2 Site Description 2
1.3 Proposed Activity 2
2 EXISTING ENVIRONMENT 4
2.1 Rainfall 4
2.2 Surface Water Bodies and Catchment Boundaries 4
2.3 Geology and Hydrogeology 5
2.3.1 Regional Geology and Hydrogeology 5
2.3.2 Local Shallow Lithology 6
2.3.3 Hydrogeological Investigations 8
2.3.4 Local Hydrogeology 9
2.4 Hydrogeological Parameters 11
2.5 Catchment Water Balance 12
2.5.1 Recharge 12
2.5.2 Groundwater and Surface Water Flows 12
2.6 Surrounding Groundwater Users 12
3 DRAWDOWN EFFECTS OF STORMWATER WETLANDS 14
3.1 Introduction 14
3.2 Analytical Pit Modelling Methodology for Stormwater Wetlands 14
3.3 Drawdown Effects of Stormwater Wetlands on Groundwater 15
3.4 Drawdown Effects of Stormwater Wetlands on Surface Water 18
4 DRAWDOWN EFFECTS OF WASTEWATER INFRASTRUCTURE 19
4.1 Introduction 19
4.2 Drawdown Effects of Wastewater Pump Station Installation 19
4.2.1 Modelling Methodology 19
4.2.2 Results 20
4.3 Drawdown Effects of Wastewater Pipeline Trenching 20
4.3.1 Modelling Methodology 20
4.3.2 Results 21
5 GROUNDWATER EFFECTS OF EARTHWORKS AND DRAINAGE 23
5.1 Introduction 23
5.2 Groundwater Inflows to Permanent Cuts 23
5.3 Groundwater Drawdown Effects of Subsoil Drainage 24
6 WATER SUPPLY 25
6.1 Introduction 25
6.2 Modelling Methodology 25
6.3 Results 25
7 GROUNDWATER EFFECTS ASSESSMENT 28
7.1 Effects Overview on Catchment Water Balance 28

7.2 Dewatering Effects Assessment 28
7.2.1 Effects on Nearby Bores 29
7.2.2 Effects on Nearby Surface Water Features 29
7.2.3 Effects on Natural Wetlands 29
7.2.4 Effects on Ground Subsidence 29
7.3 Anticipated Implications for Water Quality 29
8 MONITORING AND MITIGATION 30
8.1 Groundwater Monitoring 30
8.2 Surface Water Monitoring 30
9 CONCLUSIONS 31
10 REFERENCES 32

Figures

Figure 1: Harlow Lifestyle Village Masterplan (BBO, August 2026) 1
Figure 2: Existing Wetlands, Proposed Stormwater Wetlands, Wastewater Infrastructure and Critical Cut Inflow Location 3
Figure 3: Rainfall Record at Puniu River 4
Figure 4: Geological Setting of the Hamilton Basin (Lowe, 2021) 5
Figure 5: Underlying Geological Units 7
Figure 6: Piezometer Groundwater Levels 9
Figure 7: Piezometric Surface 10
Figure 8: Locations of Groundwater Bores and Consented Groundwater Takes 13
Figure 9: Projected Drawdown from Wastewater Pipeline MH 4B-2 to MH 4B-3 21
Figure 10: Projected Groundwater Inflows Over Time into a 20 m Trench in Pipeline MH 4B-2 to MH 4B-3 22
Figure 11: Projected Drawdown for a Range of Parameter Scenarios at 100 m from the Site Bore ID 70_1237 Location 26

Tables

Table 1: Typical Lithological Profile 6
Table 2: Piezometer Construction Details 8
Table 3: Manual Static Water Level measurements in Each Piezometer (20 July 2026) 8
Table 4: WGA Hydraulic Test Results Summary 11
Table 5: Hydraulic Conductivities of the Harlow Lifestyle Village Site 11
Table 6: Nearby Groundwater Takes 12
Table 7: Design Details Stormwater Wetlands 16
Table 8: Projected Effects of Proposed Stormwater Wetlands 17
Table 9: Existing and Design Elevations of Stormwater Wetlands Compared to Neighbouring Stream/Pond Heights 18
Table 10: Design Details Wastewater Pumping Stations 20

Table 11: Projected Effects of Wet Well Excavation 20
Table 12: Input Parameters for Wastewater Pipeline Analytical Modelling 21
Table 13: Projected Distance of Influence from Wastewater Pipeline 21
Table 14:Groundwater Inflows to Permanent Cuts 23
Table 15: Modelled Pumping Induced Drawdown for Bores Within a 1 km Radius of the Site After 120 Days of Abstraction at 100 m^3/day 27

Appendices

APPENDIX A LITHOLOGICAL LOGS
APPENDIX B GROUNDWATER LEVELS AND EXISTING BORE USERS
APPENDIX C HYDRAULIC TESTING REPORTS
APPENDIX D DRAWDOWN CALCULATIONS FOR STORMWATER WETLANDS
APPENDIX E DRAWDOWN CALCULATIONS FOR WASTEWATER INFRASTRUCTURE

1 INTRODUCTION

1.1 Background

Te Awamutu Developments Ltd is planning to establish a Retirement Living Complex, The Harlow Lifestyle Village, on a site at 2025 Ohaupo Road (the Site), approximately 200 m north of the Te Awamutu urban boundary.

The Site is an east-west oriented irregular rectangle, of approximately 25.78 ha (Figure 1). The landform drops from an elevation of 84 m above mean sea level (RL) at Ohaupo Road on the Eastern boundary to 46 m RL on the Western boundary.

The Site is dominated by rolling and undulating hills, dissected by a westerly draining watercourse, that forms a dendritic valley pattern. The valley floor widens in the western part of the Site, where an area of approximately 1 ha is occupied by Kahikatea Forest.

The Project is to enable the development of a community for senior living in Te Awamutu, providing approximately 414 dwellings/units (including single-storey houses, duplex units and apartments), and a 100-bed care facility. The development will also feature communal facilities and a neighbourhood centre, riparian planting, wetland restoration, three waters services infrastructure, and transport infrastructure. The project is designed to cater specifically to an ageing population, addressing a critical need for retirement housing and care capacity.

The development will proceed in alpha-numeric order as set out in Figure 1, notwithstanding that some downstream infrastructure that lies within latter stages will need to be constructed at the time that the earlier stages require it. General construction timing is expected to be as follows:

- Stages 1A and 1B are programmed to commence earthworks and civils in Q1–Q2 2027.
- Subsequent stages are programmed at approximately nine-month intervals between stage commencements.

WGA understands that the development will connect to the existing Council water supply and wastewater network (*'Maven'*, May 2026).

Te Awamutu Developments has engaged Wallbridge Gilbert Aztec (WGA) for technical services in relation to the substantive application for the Harlow Lifestyle Village under the *'Fast Track Approvals Act, 2024'* (FTAA). This assessment of hydrogeological effects details the technical assessment completed by WGA.



Figure 1: Harlow Lifestyle Village Masterplan (BBO, August 2026)

1.2 Site Description

Ohaupo Road runs along the eastern boundary of the Site. The Site then drops in a westward direction from the Ohaupo Road ridgeline. The Site comprises hilly terrain dissected by a westerly flowing stream that occupies a narrow valley which broadens across the western half of the Site. The stream headwaters are split into two branches, both of which contain man-made ponds. Thirteen individual Natural Wetlands have been identified across the Site, Figure 2 (Phoenix Ecology Ltd, 2026).

The land is currently used for cattle grazing, with a dwelling located at the western entrance. The dwelling and farm buildings are situated on a relatively flat area of land along the Ohaupo Road ridge.

Hedges define several paddock boundaries across the Site. Established trees surround the man-made ponds, occupy parts of the wetland areas, and line sections of the stream channel. Approximately 1 ha of remnant Kahikatea Forest occupies flat, low-lying land at the western end of the Site.

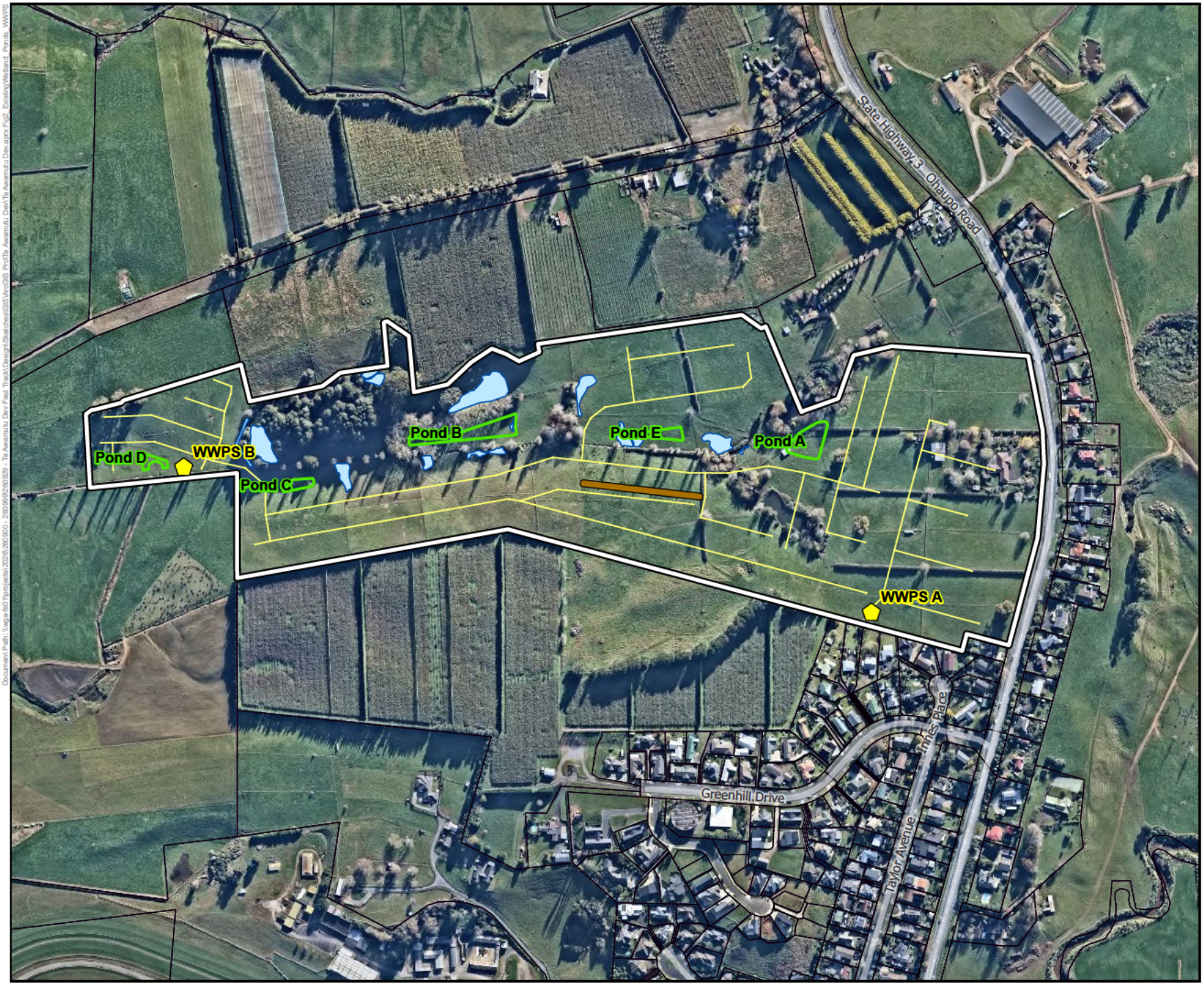
Land immediately west of the Site comprises flat, low-lying terrain used for grazing. Land to the north and south is used for a mix of purposes, including grazing and kiwifruit orchards. Residential dwellings adjoin part of the southern boundary and are also located on the opposite side of Ohaupo Road. The landform drops rapidly on the eastern side of Ohaupo Road to a stream at 45 m RL elevation.

1.3 Proposed Activity

For the Harlow Lifestyle Village five Wetlands are proposed to be constructed to manage stormwater. Additionally, two Wastewater Pump Stations (WWPS) and site-wide Wastewater Pipeline Infrastructure need to be constructed to support the development, Figure 2.

The proposed activity has three main groundwater-related components that have the potential to influence catchment water balances:

1. **Short-term temporary dewatering during construction earthworks**, for proposed Stormwater Wetlands and Wastewater Infrastructure (site-wide pipeline and two Wastewater Pumping Stations). Dewatering is anticipated to be localised and temporary, with groundwater managed and discharged in accordance with the Site erosion and sediment control framework and any relevant consent requirements.
2. **Long-term groundwater interception and diversion to the proposed unlined Stormwater Wetland**, which will act as a persistent hydraulic feature within the developed catchment.
3. **Long term groundwater interception and diversion by sub-soil drains**, should permanent cuts and retaining structures intercept groundwater**.**





LEGEND

- Project Area
- Existing Wetland
- Stormwater Pond
- Retaining Wall
- Wastewater Pump Station
- Wastewater Pipeline Design

N

0 60 120 180 m

Scale 1:6,000 @ A4

Coordinate System: NZGD 2000 Mount Eden Circuit

WGA

Figure **2**

Harlow Lifestyle Village

Existing Wetlands, Stormwater Ponds, Wastewater Infrastructure & Cut Inflow Location

Disclaimer While all reasonable care has been taken to ensure the information contained on this map is up to date and accurate, no guarantee is given that the information portrayed is free from error or omission. Any relevance placed on such information shall be at the risk of the user.

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Document Path: \\wga-fs01\projects\2025\3903900 - 3903999\3903929 - Te Awamutu Dev Feas Track\Design\Sketches\GIS\ArcGIS Pro\Te Awamutu Dev\Te Awamutu Dev.aprx Fig2_ExistingWetland_Ponds_WWPS

2 EXISTING ENVIRONMENT

2.1 Rainfall

Rainfall rates are recorded on a 15-minute basis at Puniu River - Bartons Corner (Pokuru Road Bridge) rainfall gauge, which is located approximately 5 km to the southwest of the proposed development. The available record spans from January 2010 to the present (Figure 3). Analysis for the Puniu River rainfall data indicates that during this period:

- The long-term mean annual rainfall was approximately 1,134 mm
- The peak daily rainfall recorded was 162 mm on 25 May 2026

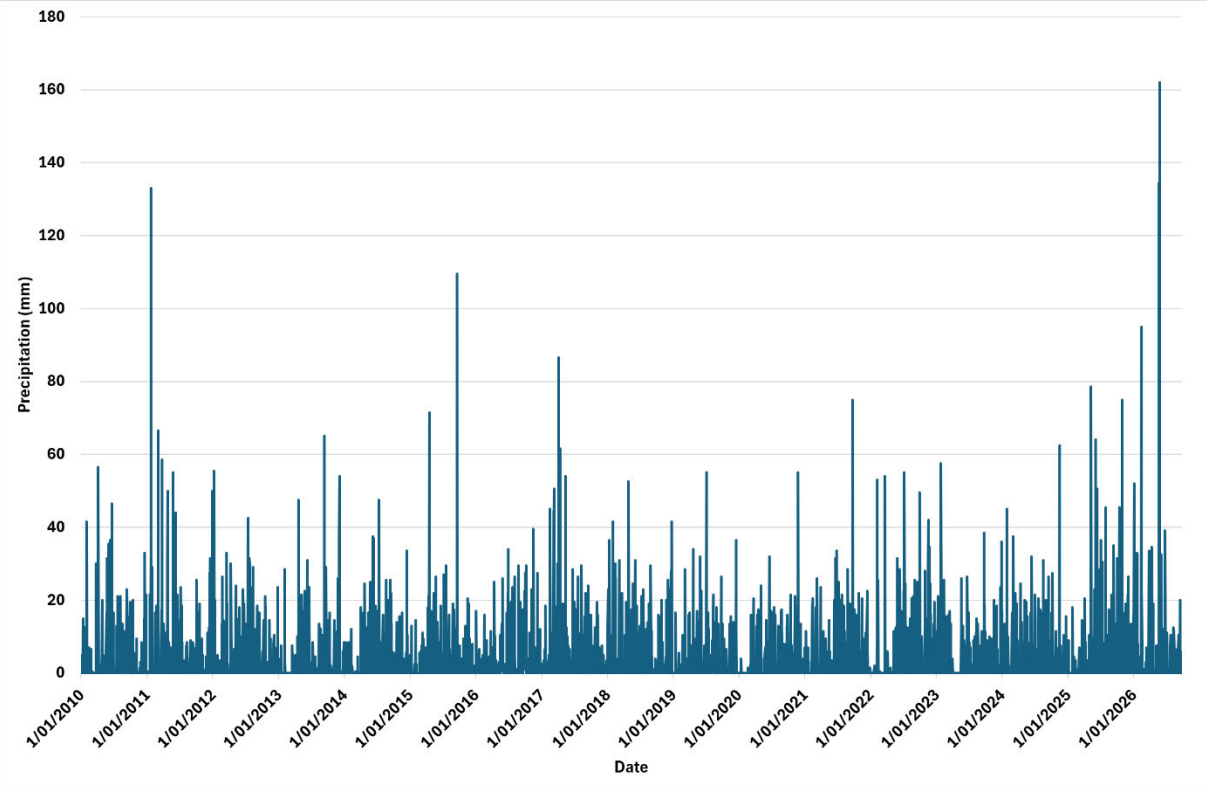


Figure 3: Rainfall Record at Puniu River

2.2 Surface Water Bodies and Catchment Boundaries

The Site is located in the Waipa Catchment area, within the Mangapiko sub-catchment, which is approximately 307 km^2 in area, and incorporates all of Te Awamutu township and surrounding rural land to the east and west of the Site.

The watercourse running through the Site extends south and westward from the western Site boundary to link with the Mangapiko Stream 1.6 km south of the Site. The Mangapiko Stream flows into the Waipa River, approximately 11 km to the west of the Site.

WGA has been advised that stormwater from the Site will be discharged into planned Stormwater Wetlands that will connect to the existing waterways and ultimately to the Mangapiko Stream.

The head of the Mangapiko Stream Tributary that runs through the Site, originates from seepage at two branches at approximately 62 m to 67 m RL in the eastern part of the Site. A series of man-made lakes have been formed by damming the watercourse in both branches of its headwaters. The lakes on the northern branch lie on the Site boundary. Where the branches meet, the stream then runs along a highly modified channel from east to west along the base of the valley. Several wetlands originating as seepages were observed in hills of the Site; these drain into the main stream channel.

The low-lying areas in the western end of the Site are occupied by permanent wetlands, and a remnant of Kahikatea Swamp. Thirteen natural wetlands have been identified on the Site (‘*Phoenix Ecology’*, 2026) many of these comprise localised areas of seepage on the lower reaches of the hills (Figure 2).

Several drains have been constructed running north to south and east-west in the low-lying area in the west of the Site. These drains were dry during the WGA site visit in May 2026. The BTW Wetland Delineation Assessment (‘*BTW*’, 2022) noted observations of several underground drainage pipes throughout the Site, discharging to the central stream.

2.3 Geology and Hydrogeology

2.3.1 Regional Geology and Hydrogeology

The Hamilton Basin, in which the proposed Harlow Lifestyle Village is located, is a large tectonic basin centred on Hamilton City with an area of approximately 2,000 km2 and traversed by the Waikato River. The basin is surrounded by ranges of Mesozoic (Manaia Hill Group) and Tertiary age (Te Kuiti and Waitemata Groups) rocks. At depth, basement greywacke underlies the sedimentary deposits that infill the basin (‘*Edbrooke*’, 2005).

The basin is infilled with Tauranga Group alluvial sediments, dating from the Pliocene to the middle Holocene. The Tauranga Group sediments are up to 300 m thick and include gravels, sands, silt, muds and peats of fluvial, lacustrine and distal ignimbritic origin.

Underlying the low hills within the basin are older ignimbrites, tephra air-fall deposits and alluvium of the Walton Subgroup (‘*Lowe’*, 2021; Figure 4).

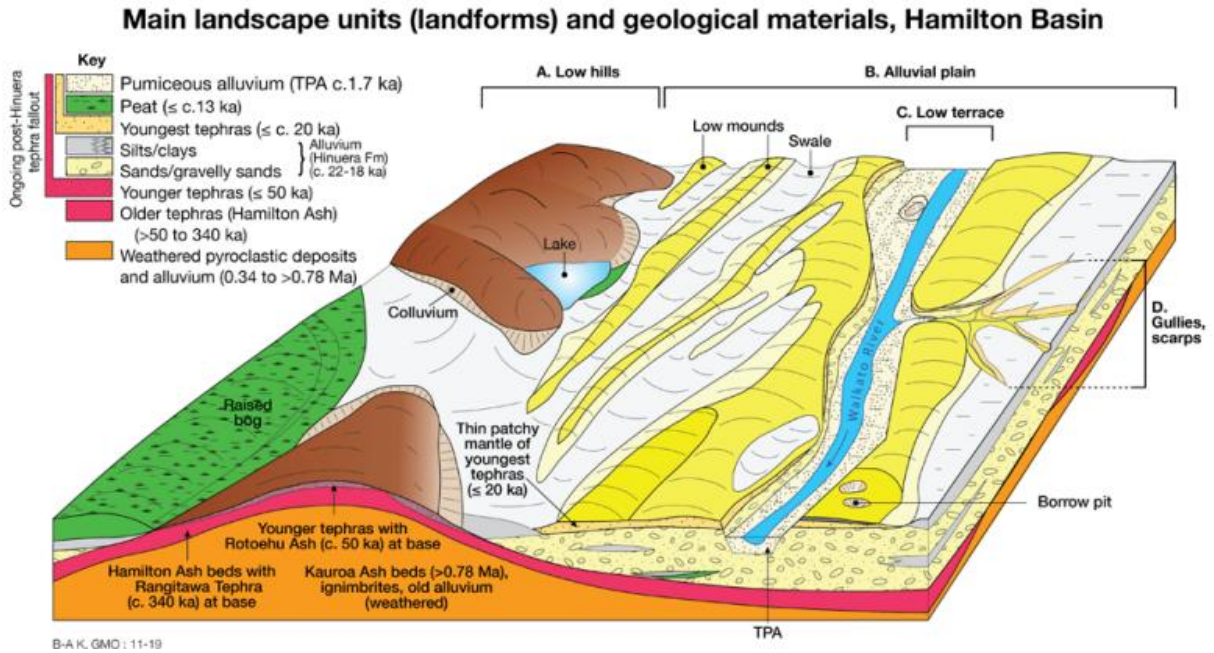


Figure 4: Geological Setting of the Hamilton Basin (Lowe, 2021)

The younger Hinuera Formation of the Tauranga Group underlies much of the Hamilton basin. This formation was deposited by braided river systems of the Waikato River, initiated by the supply of large volumes of sediment from volcanism in the Taupo Volcanic Zone (*'Petch and Marshall'*, 1988).

The Hinuera Formation contains the aquifers used most extensively across the Hamilton Basin for water supplies. Within this formation, the most productive aquifers consist of well sorted coarse sands and gravels. Discontinuous sequences of rhyolitic and pumiceous gravelly sands and gravels are interspersed with pumiceous silt, clay and peat. Lithological variability generally results in a number of zones of higher permeability within each of the formations rather than a single, continuous aquifer (*'Schofield'*, 1972). The upper layers contain perched aquifers which can dry out over the summer period and will drain to the closest gully system. Low lying areas comprise younger Holocene Swamp Deposits of the Tauranga Group, such as the Piako Subgroup, locally consisting of peat.

The Geology of the Waikato Area Map (*'Edbrooke'*, 2005) indicates that the hills of the Site are underlain by Walton Group sediments, while the stream valley is underlain by Hinuera Formation alluvium (Figure 5).

2.3.2 Local Shallow Lithology

Geotechnical investigations have been carried out by BTW in 2022 and Titus in 2026.

The BTW investigations comprised 29 Hand Augers (HA) to between 1.5 m and 2 m below ground level (bgl) and ten Cone Penetration Tests (CPT) to 20 m bgl, with one test terminated at 8 m bgl.

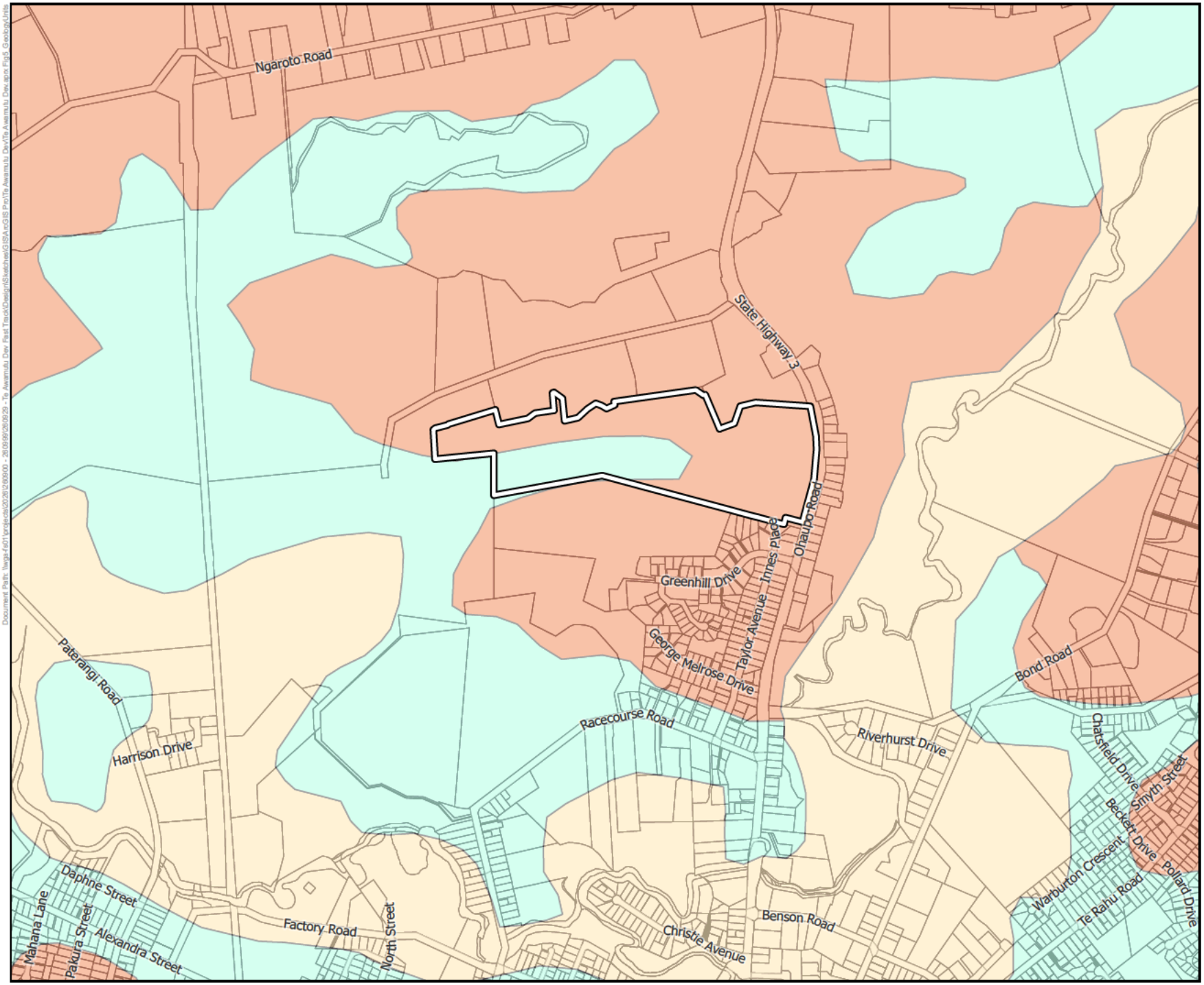
The Titus investigation comprised 14 HA's, five Test Pits (TP), and six CPT's. The ground conditions encountered are summarised in a typical lithological profile in Table 1.

Localised areas of the Site may be underlain by Recent Alluvium, Colluvium or Fill. Recently deposited alluvium is likely to be present in proximity to the stream channel. Localised areas of colluvial soil are likely to be present at the base of the steeper hills. Uncontrolled fill may be present where the stream and waterways have been modified, and in other localised areas around the Site.

Table 1: Typical Lithological Profile

UNIT	INFERRED SOIL DESCRIPTION	DEPTH TO TOP LAYER (m bgl)	LAYER THICKNESS (m)
Topsoil		0	0.1 to 0.3
Uncontrolled Fill(1)	Firm to very stiff clayey SILT, minor sand and gravel.	0	0 to 2
Hamilton Ash Formation	Stiff to very stiff Clayey SILT, with trace to some sand.	0.2 to 0.5	2.0 to 6.1
Walton Subgroup	Very stiff SILT, with clay and fine sand. With beds of medium dense SAND and silty SAND.	0.2 to 5	20+
Recent Alluvium(2)	Soft to stiff sandy SILT and clayey SILT.	0	0 to 1
Hinuera Formation(3)	Firm to very stiff clayey SILT, organic SILT, and SILT. With beds of loose to medium dense, medium to coarse SAND.	0.5 to 3.5	10+

Notes:
(1) Fill in localised areas only.
(2) Recent Alluvium present in proximity to the stream channel.
(3) Hinuera Formation present in the lower lying areas.





LEGEND

Project Area

Geologic Unit

(Holocene) Alluvium

Hinuera Formation (Late Pleistocene) river deposits

Walton Subgroup (Early Pleistocene to Middle Pleistocene) river deposits and ignimbrite

N

0 150 300 450 m

Scale 1:15,000 @ A4

Coordinate System: NZGD 2000 Mount Eden Circuit

WGA

Figure **5**

Harlow Lifestyle Village

Underlying Geological Units

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2.3.3 Hydrogeological Investigations

In July 2026 WGA oversaw the installation of three piezometers at the Site (Figure 7). An overview of piezometer as-built layouts is provided in Table 2. One of the piezometers was installed at a proposed pumping station location, two of the piezometers were installed to target proposed cuts and provide an overview of the groundwater surface beneath the hills of the Site.

Drilling provided sediment core samples for each drilling location. Samples were logged on-site by WGA in general accordance with NZGS guidelines for logging (*'NZGS'*, 2005). Drill logs are provided in Appendix A. PZ26-01 and PZ26-02 were drilled to 15 m depth, then backfilled to the base of the piezometer screen.

Table 2: Piezometer Construction Details

PIEZOMETER	DEPTH (m bgl)	CASING DIAMETER (mm)	SCREEN LENGTH (m)	EASTING (NZTM)	NORTHING (NZTM)	ELEVATION (m RL)
PZ26-01	10.5	40	1.5	1804345.8	5792474.0	70.4
PZ26-02	12.5	40	1.5	1804395.2	5792773.4	75.0
PZ26-03	8.5	40	1.5	1804120.4	5792754.9	66.4

Groundwater levels were recorded by WGA manually from the installed piezometers on 20 July 2026 (Table 3). Water level measurements were also made by Titus at numerous investigation sites in May 2026, and BTW during investigations in 2022. Recorded groundwater level measurements ranged from 1.7 to 9.95 m bgl (refer to Table B1 in Appendix B).

Table 3: Manual Static Water Level measurements in Each Piezometer (20 July 2026)

PIEZOMETER	STATIC WATER LEVEL (m bgl)	STATIC WATER LEVEL (m RL)
PZ26-01	7.52	62.88
PZ26-02	9.95	65.05
PZ26-03	5.05	61.35

Downhole continuous groundwater level monitoring has been installed in all three piezometers on 27 July 2026 to record fluctuations between dry and wet periods. The data was downloaded on 17 September 2026. The readings are shown in Figure 6.

The highest groundwater level experienced during monitoring so far was 5.0 m bgl in the PZ26-03 Piezometer. The groundwater levels were relatively stable over the monitoring period, with only small, gradual fluctuations rather than distinct rises following individual rainfall events, which is typical of a low permeability unconfined aquifer. Water levels fluctuated by up to 0.2 m during the monitoring period.

WGA recommends ongoing automated level monitoring at 15-minute intervals with downloads every three months. This information can be used for detailed design.

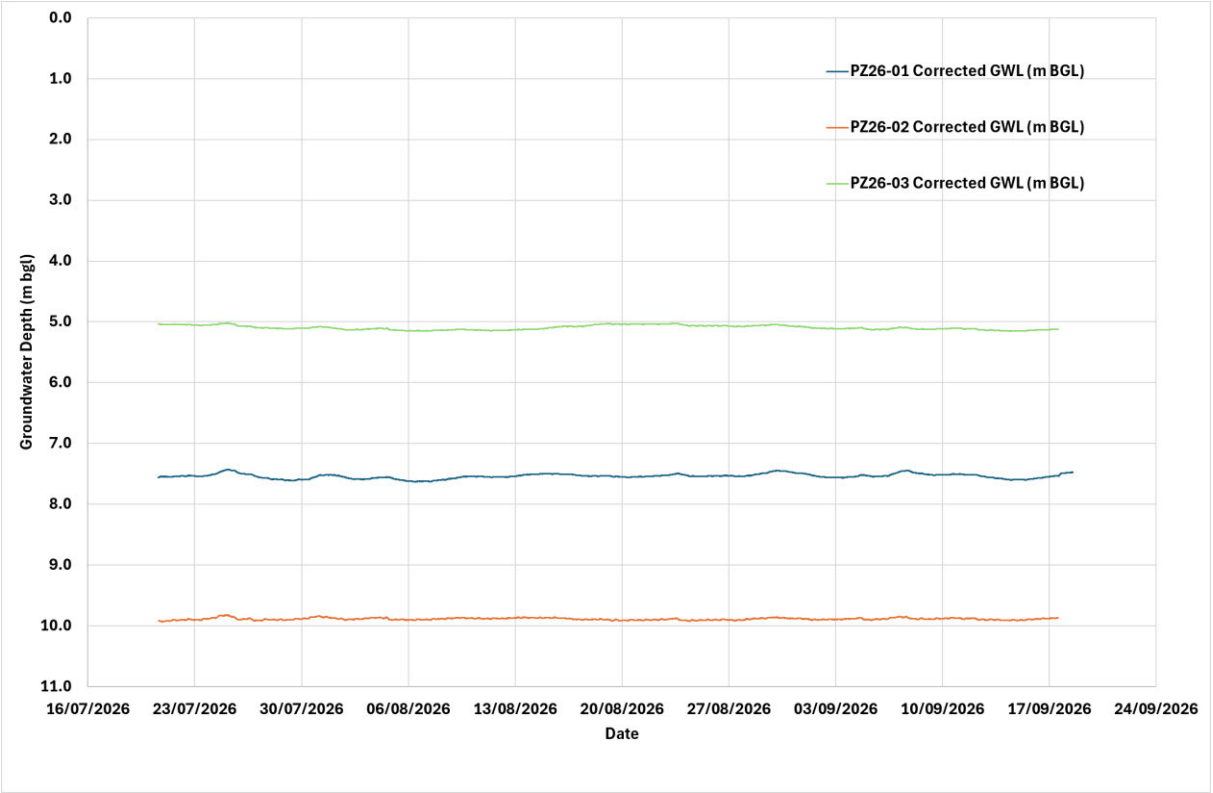
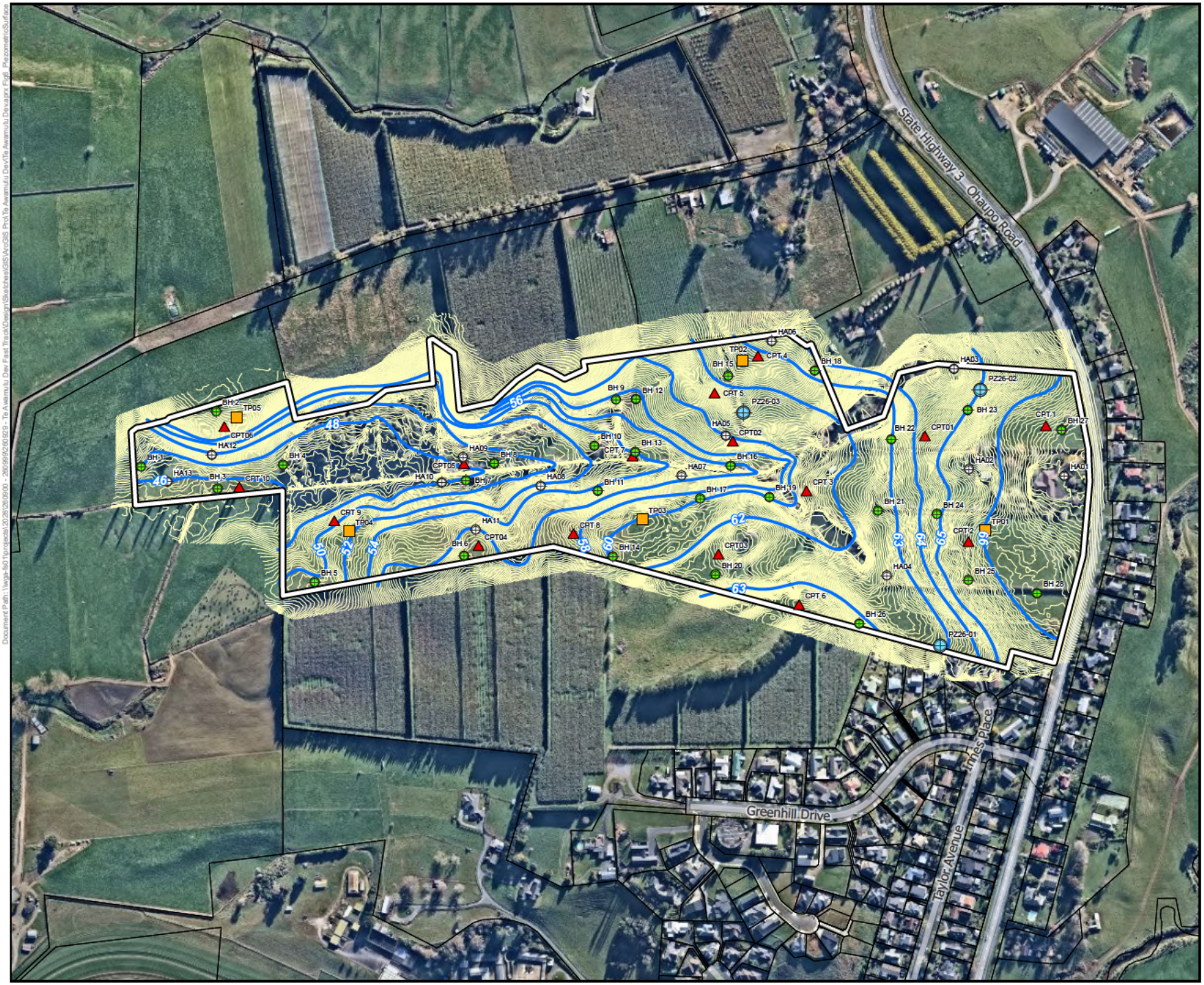


Figure 6: Piezometer Groundwater Levels

2.3.4 Local Hydrogeology

Site observations and the geotechnical investigations undertaken in the area between 2022 and 2026 were used to estimate a preliminary winter piezometric surface (Figure 7). WGA notes that much of the geotechnical groundwater data used to develop the piezometric surface was collected during different investigation periods and therefore is not temporally consistent. Consequently, the surface created is based on WGA's interpretation of the data and general understanding of the Site's hydrogeological setting.

Groundwater recharges on the upper slopes, then discharges to the wetlands, drains, man-made ponds and the Mangapiko Stream tributary running through the base of the valley. At the eastern edge of the Site the groundwater is likely to flow eastward to the Mangapiko Stream to the east of Ohaupo Road.





LEGEND

- Project Area
- Piezometric Surface Contours (m)
- Existing Contours (m)

Geotechnical Test Location

- CPT
- BH
- HA
- PZ
- TP

N

0 60 120 180 m

Scale 1:6,000 @ A4

Coordinate System: NZGD 2000 Mount Eden Circuit

WGA

Figure 7

Harlow Lifestyle Village

Piezometric Surface

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2.4 Hydrogeological Parameters

Falling and rising head tests were undertaken on each of the piezometers installed in July 2026 to derive local aquifer and aquitard hydraulic parameters. Data from these hydraulic tests were analysed using the Bouwer and Rice (1976) method. The AQTESOLV Pro v4.5 software package from HydroSolve Inc was used to analyse the datasets. The analysis sheets are presented in Appendix C.

Table 4: WGA Hydraulic Test Results Summary

PIEZOMETER ID	TESTED INTERVAL (m bgl)	SCREENED LITHOLOGY	HYDRAULIC CONDUCTIVITY[1] (m/second)
PZ26-01	9.0 to 10.5	Silty fine to coarse SAND, trace clay	4.74×10^{-8} to 2.94×10^{-7}
PZ26-02	11.0 to 12.5	Silty fine to coarse SAND, some gravel	2.75×10^{-7} to 1.05×10^{-6}
PZ26-03	7.0 to 8.5	Fine to coarse SAND, some silt, minor organic fragments	5.76×10^{-7} to 1.34×10^{-6}

Notes:
[1] All analyses completed using the Bouwer & Rice (1976) Method.

The WGA analysis results were compared to the results from infiltration testing undertaken by BTW, and decisions were made on appropriate hydraulic conductivity values for each of the lithological units listed in Table 5. For units with no data available for hydraulic test analysis, appropriate values taken from literature, and testing from similar materials, have been applied.

Table 5: Hydraulic Conductivities of the Harlow Lifestyle Village Site

MATERIAL NAME	HYDRAULIC CONDUCTIVITY RANGE (m/s)	HYDRAULIC CONDUCTIVITY ADOPTED FOR MODELLING (m/s)
Walton Subgroup	1.34×10^{-6} to 4.74×10^{-8}	1.4×10^{-6}
Hinuera Formation Silts and Clays [1,2]	1.0×10^{-4} to 2.0×10^{-8}	1.4×10^{-6}
Hinuera Formation Sands [1,3]		1.0×10^{-4}

Notes:
[1] No on site testing undertaken – Range of values derived from tests in these materials in the surrounding area and includes aquifer and aquitard layers.
[2] For modelling purposes, the same hydraulic conductivity has been adopted for the Hinuera silts and clays as for the Walton Subgroup.
[3] No site activities are expected to penetrate this unit.

2.5 Catchment Water Balance

2.5.1 Recharge

The shallow groundwater and surface waterbodies of the catchment are predominantly recharged directly by local rainfall.

WRC estimates that annual rainfall recharge (soil drainage) is 22% to 40% of rainfall in the Waikato region[1]. Based on an average annual rainfall for Waikato of 1,072 mm, this results in a recharge rate in the range of 235 mm to 428 mm. For the purposes of this assessment, a recharge value at the lower end of the estimated range (250 mm) was adopted for groundwater modelling purposes, to provide a conservative assessment of potential groundwater drawdown and associated dewatering effects. A higher recharge value of 400 mm/year has been applied when modelling construction inflows to allow for short term construction dewatering that may occur during wetter periods.

2.5.2 Groundwater and Surface Water Flows

The relatively steep slopes and low permeability soils result in a high rate of runoff on the hill slopes of the Site. Beneath the hill slopes, infiltrating rainfall flows towards the man-made ponds, stream and associated wetlands (Figure 7). Groundwater flows beneath the hills will be gradual, reflecting the low permeability of these soils. Areas of seepage are occurring from several levels on the hill slopes, some of these are likely originating from localised higher permeability layers perched on low permeability layers. The interbedded nature of the soils means that individual layers are unlikely to hold significant volumes of water.

The Ohaupo Road ridgeline is relatively narrow, and the groundwater catchment divide between flow towards the stream running through the Site and flow to the stream to the east is likely to lie beneath the eastern reaches of the Site.

In the low-lying areas, infiltrating rainfall flows towards the drains, wetlands and stream. The groundwater flows in this area will be higher than beneath the hills but will be limited by the availability of recharge in the limited local catchment.

2.6 Surrounding Groundwater Users

Water supply bores (as listed in Table B2, Appendix B) and groundwater abstraction consents within 1 km of the Site are shown in Figure 8. Groundwater in the vicinity of the Harlow Lifestyle Village is predominantly used for agricultural purposes and domestic water supplies.

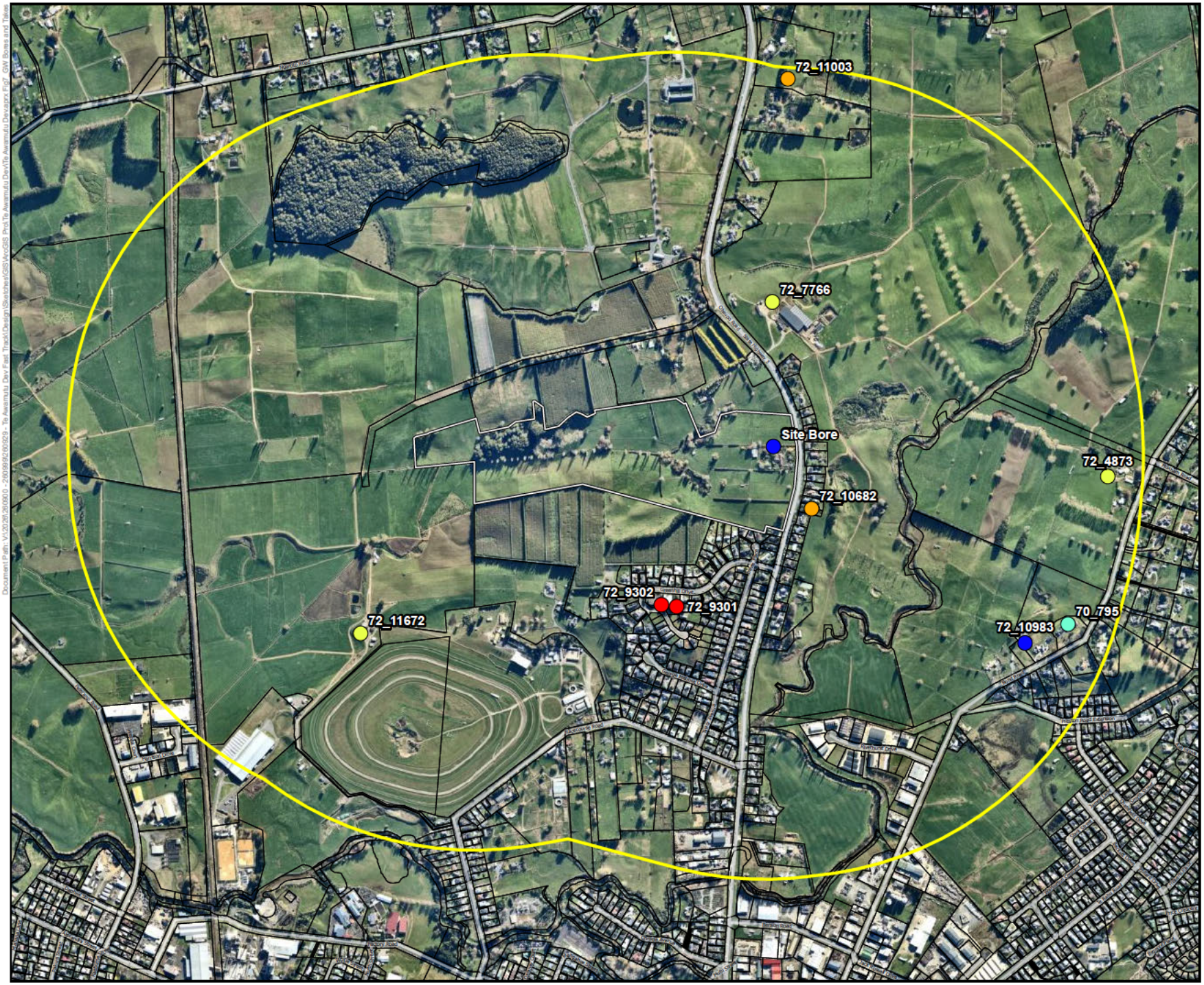
At the time of writing, according to the WRC consents database, there is one groundwater abstraction consent within 1 km of the Site. This is related to agricultural supplies. The consent is on the opposite side of the Ohaupo Ridge from the proposed Site activities to the north and east of the Site. The consent details are summarised in Table 6, and the location is shown in Figure 8.

There is an unregistered bore of unknown depth on the Site.

Table 6: Nearby Groundwater Takes

CONSENT NUMBER	USER	PERMITTED VOLUME (m^3/day)	PURPOSE
AUTH130695.01.01	Davies Landco Ltd	35.70	Agricultural Farming - Dairy

[1] https://www.waikatoregion.govt.nz/environment/water/groundwater-monitoring/flow5a-report/#:~:text=Pr%20%3D%20estimate%20of%20rainfall,percent)%20in%20the%20Waikato%20region.





LEGEND

Project Area

1 Kilometer Buffer

Nearby Bores by Depth (m)

- 25 - 50
- 50 - 75
- 75 - 100
- >100
- Unknown

N

0 150 300 450 m

Scale 1:15,000 @ A4

Coordinate System: NZGD 2000 Mount Eden Circuit

WGA

Figure **8**

Harlow Lifestyle Village

Locations of Groundwater Bores and Consented Groundwater Takes

Disclaimer While all reasonable care has been taken to ensure the information contained on this map is up to date and accurate, no guarantee is given that the information portrayed is free from error or omission. Any relevance placed on such information shall be at the risk of the user.

Note: The information shown on this map is a copyright of WGA 2026

3 DRAWDOWN EFFECTS OF STORMWATER WETLANDS

3.1 Introduction

The proposed Stormwater Wetlands will potentially intersect the existing regional groundwater levels. WGA has modelled the drawdown effects of the proposed Stormwater Wetlands using analytical methods, drawing on parameters from on-site testing (Section2.4) and the Harlow Lifestyle Village designs provided by Maven (2026). The analytical pit modelling was used to identify drawdown impacts of the Stormwater Wetlands extending to neighbouring properties and potential impacts on the existing wetlands and surface waterbodies.

Maven has verified that stormwater soakage is not required as a component of the wetland function. Wetlands A, C, D and E will be permanently lined with a GCL liner. The operational "permanent" water level in Wetland D is likely to be above the existing surrounding groundwater level. However, the wetland is proposed to be lined so will not cause any groundwater mounding.

Lining the Stormwater Wetlands limits hydraulic exchange with the surrounding groundwater, preventing long-term groundwater inflow, leakage and associated groundwater mounding or drawdown.

3.2 Analytical Pit Modelling Methodology for Stormwater Wetlands

An assessment of potential groundwater inflows to each of the planned Stormwater Wetlands has been undertaken using a published method of estimating groundwater inflows to open-cut pits (*'Marinelli & Niccoli'*, 2000). This methodology uses analogue equations to approximate horizontal groundwater inflows through the sides of the pit and vertical groundwater inflows through the floor of the pit (Appendix D).

This method adopts a conservative conceptual model in which the proposed Wetlands are assumed to be hydraulically connected to the underlying groundwater system. The analytical pit modelling applies the representative hydraulic conductivity of the Walton Subgroup and the Hinuera Formation (1.4×10^{-6} m/s) throughout the assessment.

The equations used by this methodology assume laterally radial groundwater flows toward a circular pit. In applying this methodology to the proposed stormwater wetlands, their layouts needed to be simplified by converting them to equivalent circular ponds. This has been done by taking the full footprint area of each Wetland, including forebays and rain gardens, assuming the area is circular and calculating the radius of the "equivalent" area. The effective radii for the wetlands derived using this "equivalent" calculation are presented in Table 7.

The analysis does not explicitly represent the sloping aquifer or groundwater gradient across the Site. This simplification is expected to over-predict drawdown on the upgradient side of each Stormwater Wetland and may not represent the downgradient response in detail. Therefore, the predicted radius of influence should be interpreted as a conservative screening estimate rather than a precise drawdown contour.

Design details of the proposed Stormwater Wetlands are provided in Table 7. Four of the Stormwater Wetlands (A, B, C and E) are to be constructed on the hill slopes with a combination of cut and fill construction (Figure 2). Wetland D will be constructed in the lower-lying area at the western end of the Site.

Wetland A, C, D and E will be permanently lined. For consenting purposes Wetland B has been modelled as unlined.

To assess inflows during the construction phase, the base of the Stormwater Wetlands have been modelled as dry. To assess the long-term effects of the unlined Wetland B on the groundwater regime the Stormwater Wetland has been modelled with an operational 'pit lake' representing the permanent water level. Both models used surrounding winter groundwater levels as the starting conditions to identify peak inflow rates. WGA notes that although Wetland B has an invert below the existing groundwater level the permanent water level in the Wetland is proposed to be above the surrounding groundwater levels. Therefore the long-term pit model does not cause any drawdown.

3.3 Drawdown Effects of Stormwater Wetlands on Groundwater

The groundwater inflows and drawdown effects modelled for each Stormwater Wetland are detailed in Appendix D and the results summarised in Table 8. Construction inflows assume that drawdown generated by individual Wetlands does not overlap during construction. The groundwater inflows, and the radius of groundwater drawdown influence are presented separately for construction (dry pit) and for long-term (operational permanent water level) effects in Table 8. Wetlands A, C, D and E will be lined, and Wetland B's permanent water level is above the interpreted existing groundwater level; consequently, no long-term drawdown is predicted.

The extent of groundwater drawdown is measured to the de minimis value being defined as 0.25 m drawdown. Drawdown below that magnitude is considered to be effectively indistinguishable from natural fluctuations. Natural groundwater level fluctuations are expected to be 1 to 2 m per year. The radius of influence is measured from the edge of the pit model, representing the effective edge of the Stormwater Wetland.

The drawdown effects of the Wetland construction and long-term operation (for the unlined pond) are anticipated to be minimal due to the depth to groundwater levels at the Site, the continual recharge from up-gradient, and the low permeability of the in- situ materials.

Construction inflows are in the order of 5 to 19 m^3 per day, some additional inflow may occur from localised perched groundwater, but the volume of inflow is anticipated to be easily managed with standard temporary dewatering techniques.

Table 7: Design Details Stormwater Wetlands

STORMWATER WETLAND ID	EXISTING GROUND ELEVATION (m RL)[1]	EXISTING WINTER GROUNDWATER LEVEL (m bgl)[2]	DESIGN PERMANENT WATER ELEVATION (m RL)[1]	DESIGN FOREBAY INVERT ELEVATION (m RL)[1]	DESIGN EXCAVATION DEPTH (m)	DESIGN AREA AT RIM FLOW ELEVATION (m^2)[3]	EFFECTIVE RADIUS AT OVERFLOW ELEVATION (m)[3]
A	62.2 to 68.2	1.5 to 3.5	65.4	63.9	0.6 m fill to 3.8 m cut	1,249.4	20.0
B	49.0 to 53.4	0.4 to 1.0	53.2	51.7	3.2 m fill to 1.5 m cut	2,266.7	26.9
C	47.4 to 51.6	1.0	47.6	46.1	0.4 to 4.6	996.7	17.8
D	46.2 to 48	1.0	46.2	44.7	0.2 to 2.0	977.0	17.6
E	55.8 to 61.4	0.4 to 1.5	58.7	57.2	2.2 m fill to 3.4 m cut	1,095.5	18.7

Notes:
[1] Existing and deepest design levels provided by Maven in August 2026.
[2] Estimated based on piezometric surface developed for the Site.
[3] Assuming ponded area is treated as a circle.

Table 8: Projected Effects of Proposed Stormwater Wetlands

STORMWATER WETLAND	SCENARIO	INITIAL SATURATED THICKNESS ABOVE BASIN BASE (m)	RADIUS OF DRAWDOWN INFLUENCE (m)[1,2]	CALCULATED PEAK GROUNDWATER INFLOW (m^3/day)[3]
Wetland A	Construction Dry	2.2	9.5	10.7
	Permanent Level	1.7	Nil	Nil
Wetland B	Construction Dry	1.5	6.1	9.6
	Permanent Level[3]	1.0	0	0
Wetland C	Construction Dry	3.5	17.7	16.6
	Permanent Level	3.0	Nil	Nil
Wetland D	Construction Dry	1.1	3.3	4.7
	Permanent Level	0.6	Nil	Nil
Wetland E	Construction Dry	3.7	18.3	18.4
	Permanent Level	3.2	Nil	Nil

Notes:
[1] Radius of influence considered to be 0.25 m drawdown. Taken from edge of effective radius of Wetland.
[2] Drawdown extents and inflows do not take account of changes in permeability such as Walton Subgroup sediments underlying the Hinuera Formation.
[3] Long term effects assume Stormwater Pond B is unlined.

3.4 Drawdown Effects of Stormwater Wetlands on Surface Water

Wetlands C and D are expected to temporarily cause stream depletion effects during construction due to their design base levels being lower than the water level of the adjacent stream (Table 9). Groundwater that would otherwise discharge to the stream will be captured by construction dewatering. To manage this effect, groundwater intercepted during construction dewatering shall be discharged to the receiving stream upstream of the works, to maintain surface water flows and mitigate temporary stream depletion effects.

Interception of shallow groundwater contributions to the neighbouring drains/stream baseflow is also anticipated during construction of the remaining stormwater wetlands; however, to a lesser extent because the Stormwater Wetlands do not extend deeper than the drains/stream. Any intercepted groundwater from these works will be returned to the local drainage network or stream.

The effect on the existing man-made ponds on the northern boundary by the construction of Stormwater Wetland A is also anticipated to be minimal because the wetland does not extend to the level of the pond.

The Stormwater Wetlands will all be constructed down gradient of the existing Natural Wetlands, and therefore effects on the groundwater flows to these wetlands is anticipated to be minimal.

Table 9: Existing and Design Elevations of Stormwater Wetlands Compared to Neighbouring Stream/Pond Heights

STORMWATER WETLAND	EXISTING GROUND ELEVATION (m RL)[1]	FOREBAY INVERT ELEVATION (m RL)[(1)]	DESIGN EXCAVATION DEPTH (m)	DESIGN PERMANENT WATER ELEVATION (m RL)	STREAM/POND WATER LEVEL (m RL)	DEPTH FROM DESIGN INVERT TO STREAM/POND WATER LEVEL (m)
A	62.2 to 68.2	63.9	0.6 m fill to 3.8 m cut	65.4	63.0	0.9
B	49.0 to 53.4	51.7	3.2 m fill to 1.5 m cut	53.2	50.6	1.1
C	47.4 to 51.6	46.1	0.4 to 4.6	47.6	46.4	-0.3
D	46.2 to 48.0	44.7	0.2 to 2.0	46.2	45.2	-0.5
E	55.8 to 61.4	57.2	2.2 m fill to 3.4 m cut	58.7	55.8	1.4

Note:
(1) Existing, deepest design and permanent water levels provided by Maven (August 2026).

4 DRAWDOWN EFFECTS OF WASTEWATER INFRASTRUCTURE

4.1 Introduction

WGA has modelled the drawdown effect of the dewatering required for construction of two proposed Wastewater Pump Stations and the associated Wastewater Pipeline Network. The modelling used hydrogeological parameters from on-site testing (Section 2.4) and the design details provided by Maven (April 2026).

4.2 Drawdown Effects of Wastewater Pump Station Installation

4.2.1 Modelling Methodology

Two WWPS are planned for construction: WWPS A located on the southern boundary and WWPS B located at the western end of the development (Figure 2). Detailed design of the pumping stations is not currently available. Groundwater effects have been assessed based on the construction of the Wet Well, which is anticipated to be the deepest excavation required for Pumping Station construction. Details are provided in Table 10.

It is anticipated that earthworks associated with WWPS A will not encounter the groundwater table.

WWPS B will involve excavations below the groundwater table. This Pump Station is located on Hinuera Formation Alluvium, comprising clayey silts and sands, with possible peat layers to around 5 m depth, underlain by sands and silts. The Wet Well, with a depth of approximately 4.5 m, is not anticipated to penetrate through the lower permeability upper layers into the underlying sands. A hydraulic conductivity of 1.4×10^{-6} m/s has been adopted for the Hinuera soils.

Due to the limited area of the excavation, it is expected that one bore is sufficient to achieve the required dewatering for Wet Well construction. A bore drilling permit will be required if this de-watering method is used.

The excavation dewatering has been assessed using the published method of estimating groundwater inflows to opencut pits (*'Marinelli & Niccoli'*, 2000), as was applied to the assessment of the Stormwater Wetland dewatering in Section 3. Calculations are provided in Appendix E.

The pit dewatering model applied in this assessment functions on a steady-state basis, which means the equations used apply for permanent dewatering. As the wet well construction process is expected to require only a few days, the extent of the zone of influence calculated using this methodology is conservatively large.

Earthworks associated with the pump station pipework, are anticipated to be above the groundwater table.

Table 10: Design Details Wastewater Pumping Stations

WWPS ID	EXISTING GROUND ELEVATION (m RL)[1]	DEPTH TO GROUNDWATER (m)[2]	DESIGN INVERT ELEVATION (m RL)[1]	DESIGN DEPTH (m)[4]	WET WELL EFFECTIVE RADIUS (m)[3]
A	69.0	5.6	66.1	3.0	8.0
B	46.3	1.3	44.0	4.5	8.0

Notes:
[1] Existing and deepest design levels provided by Maven April 2026.
[2] Estimated based on piezometric surface developed for the Site.
[3] Assuming a wet well excavation diameter of 15 m, and dewatering bore offset by 1 m.
[4] WWPS is in an area of planned infill.

4.2.2 Results

The results of the pit dewatering modelling are summarised in Table 11.

Table 11: Projected Effects of Wet Well Excavation

WWPS ID	DEPTH TO GROUNDWATER (m)	INITIAL SATURATED THICKNESS ABOVE BASIN BASE (m)	RADIUS OF INFLUENCE (m)[1]	CALCULATED GROUNDWATER INFLOW (m^3/day)	DISTANCE TO NEAREST BUILDING (m)	DRAWDOWN AT NEAREST BUILDING (m)
A	5.6	0	0	0	29	0
B	1.3	1.5	5.0	3.2	NA	NA

Notes:
[1] Radius of influence is considered to be defined by 0.25 m drawdown.

4.3 Drawdown Effects of Wastewater Pipeline Trenching

4.3.1 Modelling Methodology

The majority of the proposed Wastewater Pipelines will be constructed through the hilly terrain areas of the Site. Construction of pipelines in these areas is considered to have a limited impact on the existing groundwater regime as the pipeline trench inverts will generally be above the existing groundwater table.

In some areas excavation of trenches will occur below the water table. Construction requires that these trenches are maintained in a dry state until backfilling starts. WGA understands that the Wastewater Pipeline is likely to be constructed in 20 m sections, and that each section will be completed within a period of 10 days. This dewatering schedule has been used as the basis of the groundwater drawdown and inflow calculations.

Drawdown extent and magnitude have been calculated for the dewatering required for construction of the wastewater pipeline network below the existing groundwater table. The calculations used hydrogeological parameters derived from on-site testing (Section 2.4) and the network design provided by Maven (Drawing nos. C520-1 to C520-12 Rev 1A, 04/2026).

An analytical modelling approach for the wastewater pipeline was used to calculate temporary groundwater drawdown curves and expected inflows to the trenches. The analytical model used equations from McWhorter and Sunada (1977), which estimate transient drawdown and groundwater inflows into trenches.

WGA understands that the Wastewater Pipelines will be constructed at varying depths throughout the Site. WGA has completed a worst-case scenario for consideration of drawdown effects, considering a section of pipeline trench (20 m trench open at a time) within the low-lying area at the west end of the Site, requiring temporary dewatering in the Hinuera formation to up to 3.75 m depth.

Input parameters for the drawdown calculations are provided in Table 12. A higher recharge value of 400 mm/year has been adopted, which is in the higher range of values as detailed in Section 2.5.1. This is a conservative higher recharge to account for potential winter conditions during construction dewatering, resulting in higher construction inflows.

Table 12: Input Parameters for Wastewater Pipeline Analytical Modelling

WASTE-WATER PIPELINE	HYDRAULIC CONDUCTIVITY (m/s)	AQUIFER SPECIFIC YIELD	TRENCH LENGTH OPEN (m)	NUMBER OF DAYS TRENCH OPEN	RECHARGE (mm/year)	TRENCH DRAWDOWN REQUIRED (m)
MH 4B-2 to MH 4B-3	1.4×10^{-6}	0.2	20	10	400	2.2

4.3.2 Results

Projected drawdown and inflows from the wastewater pipeline trenches are summarised in Table 13. Projected drawdown is shown in Figure 9. Projected groundwater inflows are shown in Figure 10.

Table 13: Projected Distance of Influence from Wastewater Pipeline

WASTEWATER PIPELINE	DISTANCE OF INFLUENCE (m)	INFLOWS (m^3/day)
MH 4B-2 to MH 4B-3	14.0	37.6

Note: Distance of influence considered to be defined at 0.25 m drawdown.

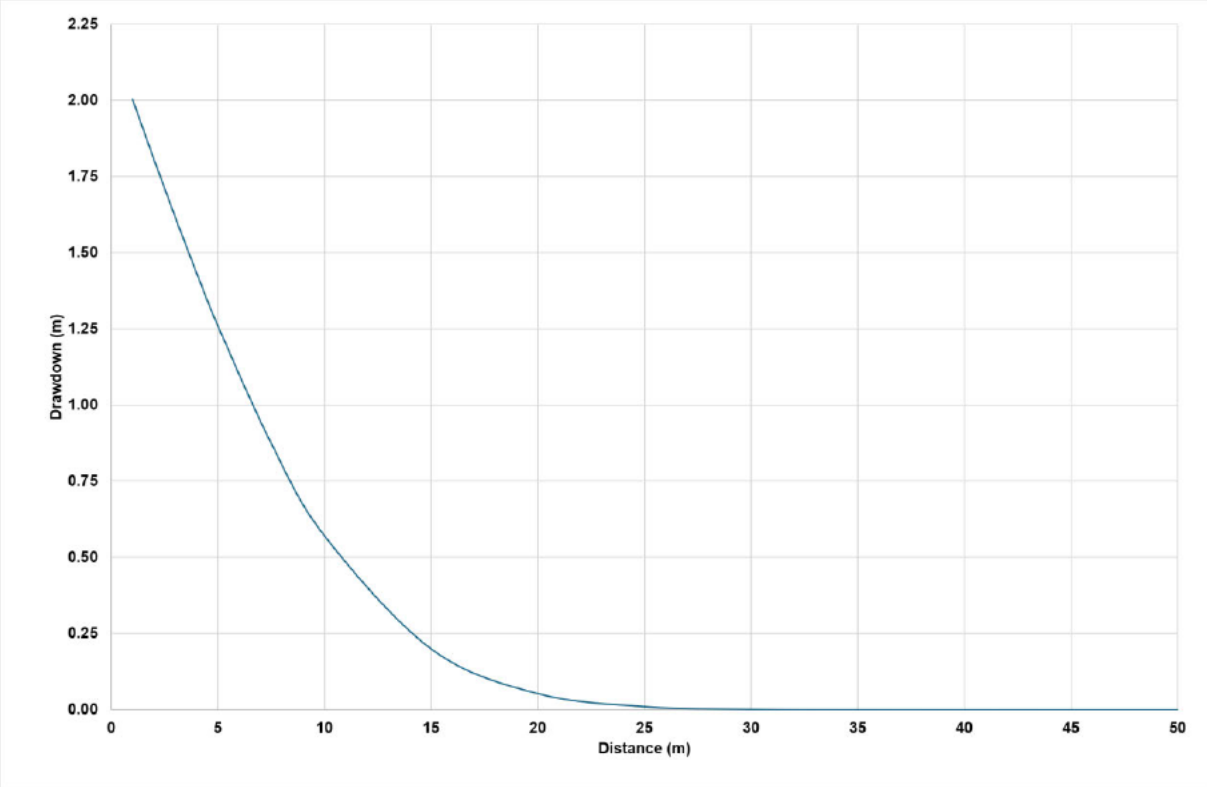


Figure 9: Projected Drawdown from Wastewater Pipeline MH 4B-2 to MH 4B-3

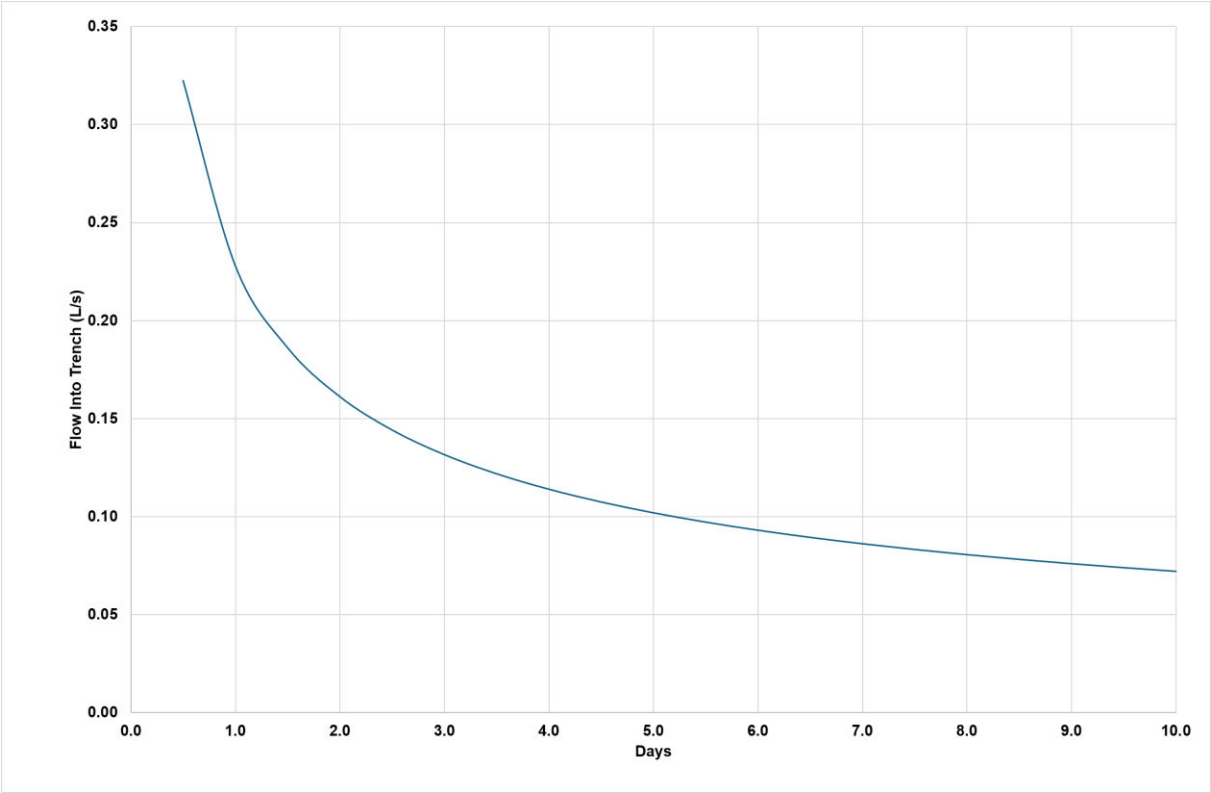


Figure 10: Projected Groundwater Inflows Over Time into a 20 m Trench in Pipeline MH 4B-2 to MH 4B-3

5 GROUNDWATER EFFECTS OF EARTHWORKS AND DRAINAGE

5.1 Introduction

The proposed development will include several cuts into the hillside to form building platforms, Stormwater Wetlands and road corridors. Cuts of up to 6 m, and retaining walls up to 4 m high, will be constructed. Subsoil drains will be installed beneath building platforms and behind retaining walls to capture and divert groundwater seepage flows to the Site stormwater system.

5.2 Groundwater Inflows to Permanent Cuts

WGA has assessed the proposed development levels relative to the piezometric surface. The majority of the proposed cuts on the slopes of the site are not anticipated to intercept the regional groundwater table. For example, piezometer PZ26-02 has a winter groundwater level of 9.9 m below the existing ground surface, and the planned cut in this area is to 3 m below existing ground surface. Localised short-term inflows from perched water tables may be encountered, but these are not expected to be substantial.

Only two areas have been identified where retaining structures may intersect the regional groundwater table (Maven Drawings C240 and C-245-1 to 6, Rev A, 2026, locations shown in Figure 2). The cuts are likely to intersect groundwater along a portion of their length. Darcy's law has been used to estimate the volume of groundwater flow through the exposed excavation.

Darcy's law estimates groundwater flow through a saturated soil or rock mass based on the hydraulic conductivity of the material, the hydraulic gradient, and the cross-sectional area available for flow. The calculation is expressed as $Q = K\,i\,A$, where Q is the groundwater flow rate, K is the adopted hydraulic conductivity, i is the hydraulic gradient, and A is the saturated area of the exposed cut face. For this assessment, the flow area was based on the length and saturated height of the retaining wall cut, and the adopted hydraulic conductivity was selected from the site-specific hydraulic testing and interpreted geological unit (Section 2.4). The calculated flow rate represents an estimate of groundwater seepage that may need to be intercepted by subsoil drainage and discharged to the Site stormwater system.

The calculation assumes that seepage is controlled by the adopted hydraulic conductivity for the intercepted unit and by the interpreted local hydraulic gradient toward the cut face. The estimate should therefore be treated as an order-of-magnitude allowance for drainage design rather than a prediction of continuous long-term discharge from all cut faces. Flows expected based on this calculation are shown in Table 14.

Table 14:Groundwater Inflows to Permanent Cuts

CUT LOCATION	LENGTH	HEIGHT (m)	EXISTING GROUNDWATER LEVEL (m bgl)	HYDRAULIC GRADIENT	HYDRAULIC CONDUCTIVITY (m/s)	FLOW (m^3/day)
1	133	3.1	1.0	0.13	1.4×10^{-6}	4.4
2	30	3.2	2.5	0.07	1.4×10^{-6}	0.8

Based on calculations at the two expected groundwater inflow areas, and allowing for perched groundwater flows, WGA consider that the maximum groundwater diversion at the site from sub-soil drainage will be less than **50 m^3/day**.

5.3 Groundwater Drawdown Effects of Subsoil Drainage

Subsoil drains may intercept shallow groundwater and perched groundwater where permanent cuts and retaining structures intersect water-bearing layers. However, due to the strong hydraulic gradient towards the valley, low aquifer permeability and continued upgradient recharge, any associated drawdown is expected to be localised and minimal.

WGA notes that intercepted groundwater will be conveyed through the stormwater system and returned to the local stream, thereby maintaining flows within the wider catchment.

6 WATER SUPPLY

6.1 Introduction

WGA understands that the development will obtain water supply from the Te Awamutu Municipal Supply Network. However, water will be required during construction for dust suppression purposes. WGA understands that volumes in the order of 100 m^3/day may be required from on-site sources based on calculations carried out by Maven, for dust suppression volumes (‘*Maven CMP’*, 2026). The on-site bore (WRC ID 70_1237), location shown in Figure 8, may be suitable for this purpose. At the time of writing the construction, yield and groundwater source for the bore has not been determined. To cover the uncertainty in bore depth and local hydraulic parameters a range of aquifer parameters based on a lower range of transmissivity values from the Hamilton Basin have been used to calculate potential effects.

The water from on-site ponds, which will subsequently be removed during the development, may also be utilised. Volumes in the order of 80 to 100 m^3/day may be pumped from these ponds prior to their infilling.

6.2 Modelling Methodology

WGA have completed an analysis of the potential effects on neighbouring bores from utilising the bore for dust suppression during construction. The published Theis (1935) model was used to calculate drawdown from a pumping well under confined aquifer conditions.

The Theis solution assumes ideal aquifer conditions (confined, homogeneous, isotropic, infinite in extent, and fully penetrating wells) and does not account for physical boundaries or heterogeneities that may limit drawdown. As a result, the method generally overestimates drawdown at nearby bores, making it a conservative approach.

A maximum daily pumping rate of 100 m^3/day has been modelled, running 24 hours per day for 120 days, to provide for dust suppression needs (Maven, email correspondence 2026).

Due to the lack of data on the existing bore a range of potential parameters was adopted in the modelling, reflecting the expected range for a confined aquifer in the Hamilton Basin:

- Transmissivity between 20 and 30 m^2/day.
- Storativity between 0.0001 and 0.001.

6.3 Results

A Monte Carlo statistical analysis has been used to estimate the range of possible outcomes based on the conservative range of parameters. The range of projected drawdowns from this analysis, at 100 m from the bore location, are illustrated in Figure 11. The projected drawdown effect on bores within a 1 km radius of the Site based on this modelling is shown in Table 15.

The available column of water for each bore was calculated by deducting the reported water level from the casing depth and reducing the available column by an additional 2 m to account for a pump at the bottom of the casing. Bores without documented casing depths or static water levels were assumed to be in the same aquifer as the Site bore. In order for an assessment to be undertaken in these bores, casing depths and static water levels were estimated.

All bores simulated under these conservative drawdown scenarios indicated a drawdown less than 10% of the available water column. As such, drawdown effects from the use of the Site production bore for dust suppression are considered less than minor.

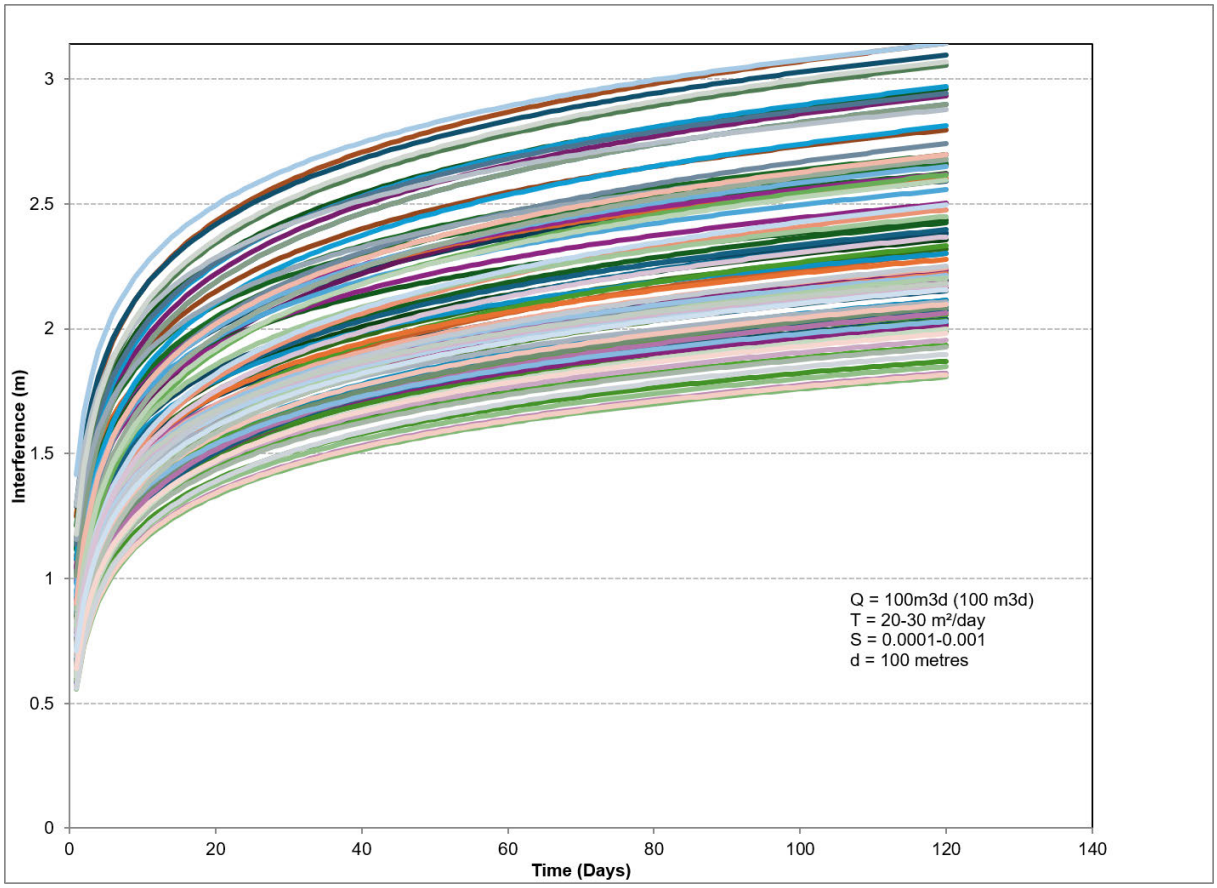


Figure 11: Projected Drawdown for a Range of Parameter Scenarios at 100 m from the Site Bore ID 70_1237 Location

Table 15: Modelled Pumping Induced Drawdown for Bores Within a 1 km Radius of the Site After 120 Days of Abstraction at 100 m^3/day

BORE ID [1]	EASTING (NZTM)	NORTHING (NZTM)	BORE DEPTH (m)	CASING DEPTH (m)	STATIC WATER LEVEL (m bgl)	DISTANCE FROM PRODUCTION BORE (m)	AVAILABLE BORE WATER COLUMN (m)	PROJECTED DRAWDOWN (m)	PROJECTED CHANGE IN AVAILABLE WATER COLUMN (%)
72_9302[2]	1804118	5792250	120	74	34.4	448	38	2.5	7%
72_9301	1804162	5792244	124	74	33	471	39	2.5	6%
72_10682	1804557	5792519	86	60	27	617	31	2.0	6%
72_7766[3]	1804455	5793118	56	46	10	669	34	1.9	6%
72_11672	1803248	5792184	60	49	9.7	861	37	1.7	5%
72_11003	1804512	5793761	86	52.5	14.9	1,224	36	1.4	4%
72_10983[4]	1805166	5792120	46	32.5	9	1,326	22	1.4	7%
70_743	1802791	5791868	188.4	100	35	1,416	63	1.3	2%
70_795	1805291	5792172	46	32.5	9	1,422	22	1.3	6%
72_4873	1805414	5792596	58	49	6	1,457	41	1.3	3%

Notes:
[1] Data obtained from the WRC database.
[2] The casing depth for 72_9302 is unknown; therefore, it was assumed to be similar to the adjacent 72_9301 bore.
[3] The static water level for 72_7766 is unknown; therefore, it was assumed to be similar to 72_11672.
[4] The bore depth, static water level and casing depth for 72_10983 was assumed to be similar to nearby bore 70_795.

7 GROUNDWATER EFFECTS ASSESSMENT

7.1 Effects Overview on Catchment Water Balance

There is a strong hydraulic gradient across the Site, and high existing run off. Groundwater has been flowing towards the Wetlands and stream.

The main groundwater-related components that have the potential to influence the catchment water balance:

1. Short-term temporary dewatering during construction of the Stormwater Wetlands and Wastewater infrastructure.
2. Long-term groundwater interception and diversion to the subsoil drains beneath building platforms and behind retaining walls.
3. Long-term groundwater interception and diversion to the proposed unlined Stormwater Wetland which will act as persistent hydraulic features within the developed catchment.

The proposed activity has only a small drawdown effect and is unlikely to affect groundwater flows. Any groundwater diversions will be returned to the local stream, to which it would naturally discharge. Therefore, the effects on groundwater are expected to be minimal.

The development represents a land use change from existing grazing farmland to residential. This land use change is expected to alter the way rainfall is divided between infiltration (recharge), evapotranspiration, and runoff. A typical residential development ratio for impervious/pervious land is approximately 70 % impervious and 30 % pervious. Rainfall recharge is expected to reduce beneath impervious surfaces relative to the existing rural state, with a corresponding increase in runoff captured by the proposed Stormwater Management System. Given the existing steeper slopes and low permeability materials the existing recharge at the site is expected to be relatively low compared to flatter properties.

In the long term, the Site water balance is expected to shift from predominantly distributed recharge and surface runoff toward a more managed regime, where a larger proportion of the rainfall is routed to engineered Stormwater Wetlands, stored within, and subsequently discharged to existing drains and the tributary to the Mangapiko Stream.

7.2 Dewatering Effects Assessment

Limited dewatering effects are expected from this development as outlined in Sections 3 and 4.

The largest stormwater construction groundwater abstraction is expected to occur for Wetland E at **18.4 m^3/day**. Pumped groundwater (where required during construction) and diverted inflows will be managed under a Water Management Plan and an erosion and sediment control plan for the project.

WGA understands that Wastewater Infrastructure is planned to be constructed progressively, based on the installation of consecutive 20 m lengths of pipeline in similar dewatered trench sections. The WWPS's will be constructed separately, with construction schedules that will probably overlap the construction schedule for the pipelines. Based on the current design, the largest groundwater abstraction for pump station construction is expected at WWPS B, at **3.2 m^3/day**. The largest abstraction for pipeline installation is expected to occur for the deepest section of pipe below groundwater, at **37.6 m^3/day** (for a 20 m section, constructed over 10 days).

Initial dewatering flow rates may need to be higher than the modelled steady-state values to achieve the required drawdown within the available construction timeframe.

7.2.1 Effects on Nearby Bores

Of the surrounding groundwater users, there is only one current groundwater take consent within 1 km of the Site (Section 2.6). The groundwater take is located at 17 m RL to 27 m RL, well below the deepest extent of this development.

If the on-site bore is used for dust suppression during construction, the effect on nearby bores is anticipated to be minimal.

There are no other drawdown effects, assessed in this report, that extend to nearby bores.

7.2.2 Effects on Nearby Surface Water Features

Where construction activities temporarily intercept groundwater that would otherwise discharge to local drains or the tributary to the Mangapiko Stream, it is recommended that the intercepted groundwater be discharged back to the receiving drainage network upstream of the works. This measure maintains continuity of surface water flows and mitigates short-term stream depletion effects during construction. With these measures in place, no sustained reduction in surface water flows or substantial changes to downstream hydrology are expected. Overall effects on stream flows are assessed to be minimal and temporary.

7.2.3 Effects on Natural Wetlands

Thirteen Natural Inland Wetlands have been identified within the Site (*'Phoenix Ecology'*, 2026). A small Wetland will be unavoidably removed as part of the proposed development (as detailed in the '*Phoenix Ecology'*, 2026 Report). As a result, any effects of groundwater drawdown on this feature are not relevant to the construction or operational assessment.

All the Natural Wetlands are upgradient of the proposed Stormwater Wetlands. Drawdown effects from the planned Stormwater Wetlands are limited to, a small extent from construction dewatering with no long-term drawdown effects. Construction dewatering water is being returned to the local drainage network, groundwater drawdown arising from construction operations is not expected to alter the hydrological regime, extent, or ecological function of the retained Natural Wetlands. Overall effects on Natural Wetlands are assessed to be de minimis.

7.2.4 Effects on Ground Subsidence

The effect of the "worst-case" drawdown associated with construction dewatering has the potential to induce minor ground subsidence, particularly where drawdown occurs within compressible near surface soils or where seasonal groundwater levels are temporarily lowered beyond naturally experienced fluctuations. The projected groundwater drawdown during construction will not extend beneath nearby housing.

7.3 Anticipated Implications for Water Quality

The land use change from grazing farming to residential land use, together with the introduction of Stormwater Wetlands, is expected to result in reduced nutrient inputs to shallow groundwater and the Mangapiko Stream Tributary over time (relative to the existing rural baseline). While construction activities can temporarily elevate sediment related risks, this risk is to be addressed through the development and implementation of an erosion and sediment control plan.

8 MONITORING AND MITIGATION

8.1 Groundwater Monitoring

Groundwater levels across the Site have been characterised using a network of three piezometers installed in July 2026. The period between installation and delivery of this report is not sufficient to capture both winter and summer groundwater conditions. Therefore, the network should be monitored over the next 12 months to define seasonal groundwater level variability. Automated monitoring should be carried out at 15-minute intervals and downloaded every three months. The levels can be used for detailed design.

The predicted groundwater drawdown effects associated with construction dewatering and long-term operation of the proposed Stormwater Wetlands are localised, temporary (during construction), and have been conservatively assessed using site-specific hydraulic parameters. Groundwater drawdown effects do not impact the availability of water at nearby groundwater abstraction bores. The long-term groundwater diversion to the unlined Stormwater Wetland, represents a redistribution of groundwater discharge within the Site catchment rather than a net loss.

The need for permanent or post-construction groundwater monitoring should be reviewed after the 12-month baseline record, detailed design, and construction-phase verification are complete. Monitoring may be discontinued where the observed groundwater response confirms the conceptual model and predicted effects, and where no persistent interception, drawdown, or change to groundwater-dependent features is identified.

8.2 Surface Water Monitoring

Surface water features within and downstream of the Site comprise farm drains and the headwaters of the Mangapiko Stream. At least some of these features are hydraulically connected to the groundwater system identified within the Site.

Potential effects on surface waters are limited to localised interception of groundwater contributions during construction dewatering for Stormwater Wetlands and wastewater infrastructure. These effects have been assessed as temporary and manageable through construction methodology and the appropriate placement of temporary water discharge points rather than ongoing operational controls.

Given the short duration of dewatering activities, the absence of predicted long-term surface water effects, and the implementation of erosion and sediment control measures under the project's construction management framework, additional surface water flow or level monitoring is not considered necessary for the operational phase of the development. Visual inspections of drains and stream reaches during construction, undertaken as part of standard environmental compliance checks, are considered sufficient to confirm expected performance.

9 CONCLUSIONS

This assessment has considered the potential groundwater effects associated with the construction and operation of the Harlow Lifestyle Village, including groundwater abstraction for dust suppression during construction, construction dewatering for Stormwater Wetlands and Wastewater Infrastructure, and the long-term groundwater diversion associated with the proposed Stormwater Wetlands and subsoil drains associated with retaining structures. The investigations, monitoring and analytical assessments undertaken indicate that the predicted groundwater effects are localised and can be appropriately managed through the proposed development design and associated management measures. Accordingly, groundwater effects associated with the proposed development are assessed as minimal.

Based on the investigations undertaken, modelling completed, and mitigation measures proposed, WGA considers that the groundwater effects associated with the construction and operation of the Harlow Lifestyle Village can be appropriately managed.

Continuous groundwater monitoring within the three on-site piezometers is recommended for a 12-month period to identify seasonal variation in groundwater levels and to validate the preliminary winter piezometric interpretation used in this assessment. The need for permanent or post-construction groundwater monitoring should be reviewed after the 12-month baseline record, detailed design, and construction-phase verification are complete. Monitoring may be discontinued where the observed groundwater response confirms the conceptual model and predicted effects, and where no persistent interception, drawdown, or change to groundwater-dependent features is identified.

10 REFERENCES

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APPENDIX A
LITHOLOGICAL LOGS



Table A1: Lithological Log for Piezometer PZ26-01

FROM (m)	TO (m)	LITHOLOGICAL DESCRIPTION
0	0.2	TOPSOIL: dark brown
0.2	2.1	Silty CLAY, brown, homogeneous, moderate plasticity.
2.1	5.2	CLAY, brown, homogeneous, high plasticity, hard.
5.12		Moist
5.2	5.8	Silty CLAY, light brown, moderate plasticity.
5.8	6.26	CLAY, minor coarse sand, minor silt, light grey, mottled orange, hard.
6.26	10.5	Silty fine to coarse SAND, trace clay, light grey/brown, low plasticity.
7.9		Wet
10.5	11.53	Sandy CLAY, some silt, grey to brown with orange mottles, moderate plasticity, hard.
11.53	12.28	Silty fine to coarse SAND, trace clay, brown, mottled orange, poorly sorted, low plasticity, tightly packed.
12.28	13.33	Fine to coarse SAND, with silt, minor gravel, brown to orange, poorly sorted, very loose.
13.33	13.6	Coarse SAND, some silt, brown, moderately sorted.
13.6	15.0	SILT, minor to some clay, minor to some fine to coarse sand, white to cream.

Table A2: Lithological Log for Piezometer PZ26-02

FROM (m)	TO (m)	LITHOLOGICAL DESCRIPTION
0	0.23	TOPSOIL, dark brown.
0.23	1.5	CLAY, minor silt, brown, moderate plasticity, hard.
1.5	3	CLAY, light brown, very homogeneous, high plasticity, hard.
3	3.04	Silty CLAY, minor fine sand, light grey/brown, mottled orange, moderate plasticity, hard.
3.04	3.2	CLAY, light brown/grey, mottled orange, with black organic fragments, high plasticity, hard.
3.2	3.25	Silty CLAY/ SILT, minor fine sand, light grey/brown, mottled orange, moderate plasticity, hard.
3.25	4.9	Silty CLAY, minor fine sand, brown, with black organic fragments, high plasticity.
4.9	5.61	Silty CLAY, some sand and gravel, light pink, with black organic fragments, moderate plasticity.
5.61	6.51	Silty CLAY, minor fine sand, trace pumiceous gravel, light pink to grey, moderate plasticity.
6.51	8.26	Silty CLAY, some sand, minor gravel, light grey to pink, mottled orange, moderate plasticity.
8.26	9.7	Silty fine to medium SAND, light brown, banded orange, with black inclusions, poorly sorted, tightly packed.
9.9		Wet.
9.7	12	Silty fine to coarse SAND, some gravel, brown, poorly sorted.
9.84		20 mm band of stiff organic silt.
10.75		Saturated.
12	13.36	SILT, some fine to medium sand, light grey, moderately well sorted.
13.36	13.94	Sandy SILT, light brown, moderate plasticity, moderately well sorted.
13.94	14.85	Medium to coarse SAND, with silt, light brown, moderately well sorted.
14.85	15	Fine SAND, some silt, light brown, well sorted, tightly packed.

Table A3: Lithological Log for Piezometer PZ26-03

FROM (m)	TO (m)	LITHOLOGICAL DESCRIPTION
0	0.2	TOPSOIL, dark brown.
0.2	3.58	Silty CLAY, minor fine sand, brown, moderate plasticity.
1.5	2.5	Soft, SILT, minor clay, minor fine sand, low plasticity.
1.5		Minor gravel.
1.8		Moist.
3.58	4.7	Silty CLAY, with sand and gravel, brown to grey (patchy), mottled and streaked orange, poorly sorted, high plasticity.
4.7	4.95	Clay, minor sand, light grey to brown, high plasticity, hard.
4.95	5.8	Silty CLAY, some sand and gravel, minor black organic fragments, brown to grey, mottled orange, moderate plasticity, hard.
5.8	6.06	Silty fine to coarse SAND, minor clay, banded brown, white, and orange, low plasticity.
5.8		Large, rounded gravel fragment.
6.06	8.5	Fine to coarse SAND, some silt, minor organic fragments, poorly sorted, loosely packed/soft.
8.5	9	Clayey SILT, some sand, trace organic fragments in bands, moderate plasticity, poorly sorted. Very stiff to hard/ tightly packed.

APPENDIX B
GROUNDWATER LEVELS AND EXISTING BORE USERS



Table B1: Groundwater Levels Recorded during geotechnical Investigations

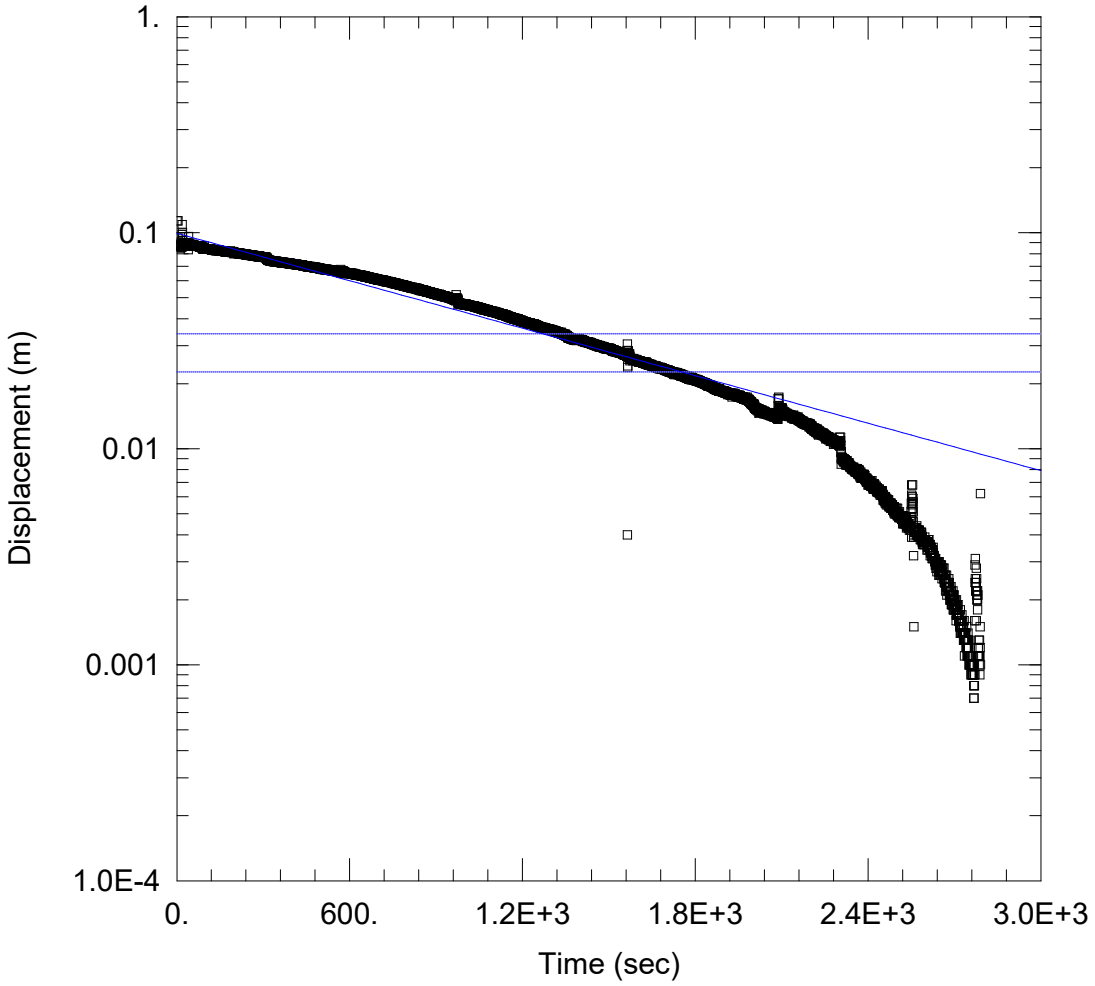
TEST ID	TEST TYPE	GROUNDWATER LEVEL (m bgl)	GROUNDWATER LEVEL (m RL)
BH 1	HA - BTW	0.9	48.1
BH 3	HA - BTW	1.0	45.0
BH 4	HA - BTW	1.4	45.6
BH 8	HA - BTW	1.6	48.4
BH 10	HA - BTW	1.1	51.9
BH 12	HA - BTW	0.4	57.6
CPT 1	CPT - BTW	Collapsed dry at 5.0	
CPT01	CPT - Titus	Collapsed dry at 6.3	
CPT 2	CPT - BTW	Collapsed dry at 3.0	
CPT02	CPT - Titus	Collapsed dry at 1.3	
CPT 3	CPT - BTW	Collapsed dry at 2.0	
CPT03	CPT - Titus	Collapsed dry at 5.3	
CPT 4	CPT - BTW	9.5	62.5
CPT04	CPT - Titus	Collapsed dry at 2.5	
CPT 5	CPT - BTW	5.0	62.0
CPT05	CPT - Titus	1.1	48.9
CPT 6	CPT - BTW	Collapsed dry at 2.0	
CPT06	CPT - Titus	Collapsed dry at 2.7	
CPT 7	CPT - BTW	Artesian flow	
CPT 8	CPT - BTW	3.5	57.5
CPT 9	CPT - BTW	2.1	49.9
CPT 10	CPT - BTW	1.0	45.0
HA03	HA - Titus	2.9	65.1
HA07	HA - Titus	0.6	55.4
HA08	HA - Titus	0.6	52.4
HA09	HA - Titus	1.4	48.6
HA10	HA - Titus	1.5	48.5
HA13	HA - Titus	1.2	45.8
TP04	TP - Titus	4.2	48.8
PZ26-01	Piezo - WGA	7.52	62.58
PZ26-02	Piezo - WGA	9.95	64.85
PZ26-03	Piezo - WGA	5.05	61.05

Table B2: Nearby Bores within 1 km of Proposed Site Activities from the WRC Database (March 2026)

BORE ID	TYPE	STATUS	EASTING	NORTHING	EXTERNAL CASING DIAMETER (mm)	DRILLED DEPTH (m)
72_9302	Well/Bore	Existing	1804118	5792250	100	120
72_9301	Well/Bore	Existing	1804162	5792244	150	124
72_10682	Well/Bore	Existing	1804557	5792519	100	86
72_7766	Well/Bore	Existing	1804455	5793118	100	56
72_11672	Well/Bore	Existing	1803248	5792184	100	60
72_11003	Well/Bore	Existing	1804512	5793761	100	86
72_10983	Well/Bore	Existing	1805166	5792120	-	-
70_743	Well/Bore	Existing	1802791	5791868	50	188.4
70_795	Well/Bore	Existing	1805291	5792172	100	46
72_4873	Well/Bore	Existing	1805414	5792596	100	58

APPENDIX C
HYDRAULIC TESTING REPORTS





WELL TEST ANALYSIS

Data Set:
Date: 07/21/26 Time: 14:13:51

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

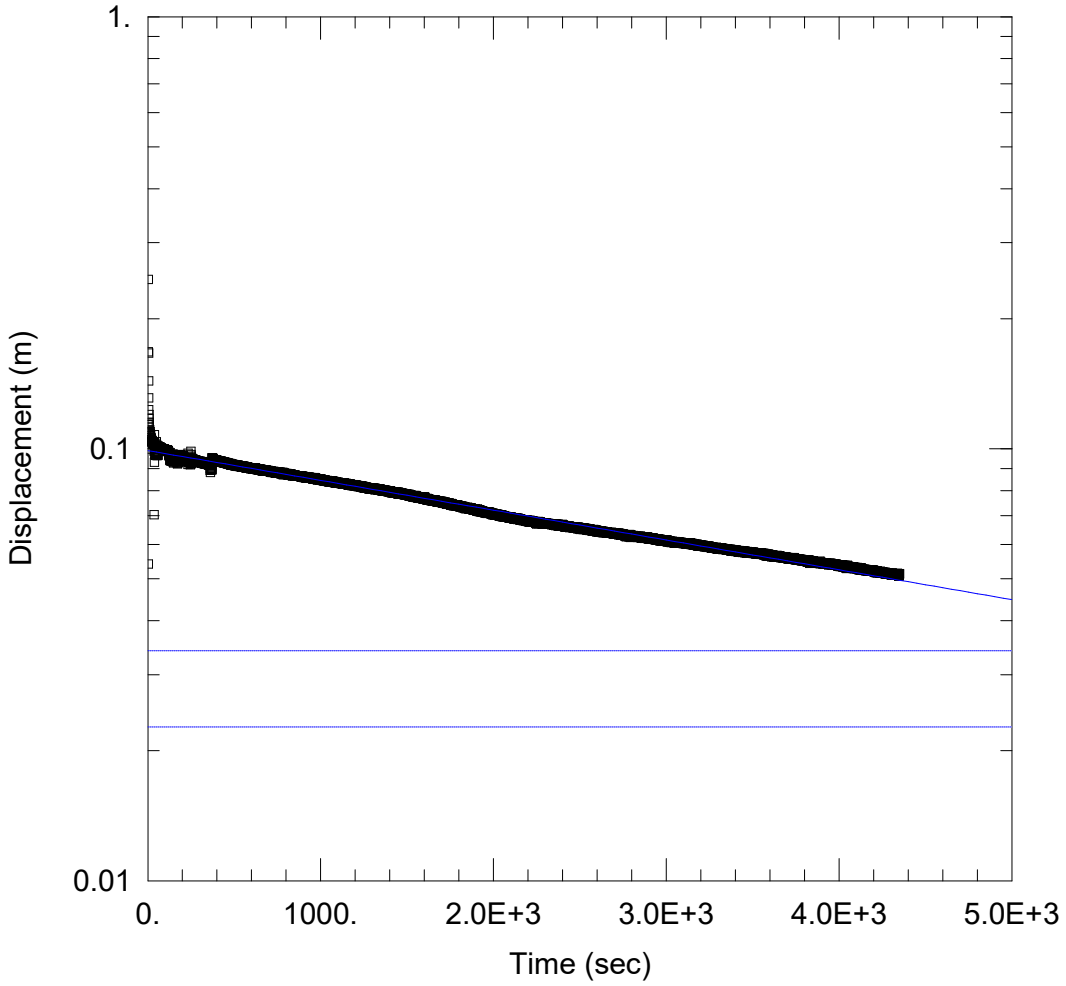
Saturated Thickness: 5.978 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (PZ26-01 FHT)

Initial Displacement: 0.1136 m Static Water Column Height: 2.978 m
Total Well Penetration Depth: 2.978 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 2.939E-7 m/sec y0 = 0.09951 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-01 RHT.aqt
Date: 07/21/26 Time: 14:19:56

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

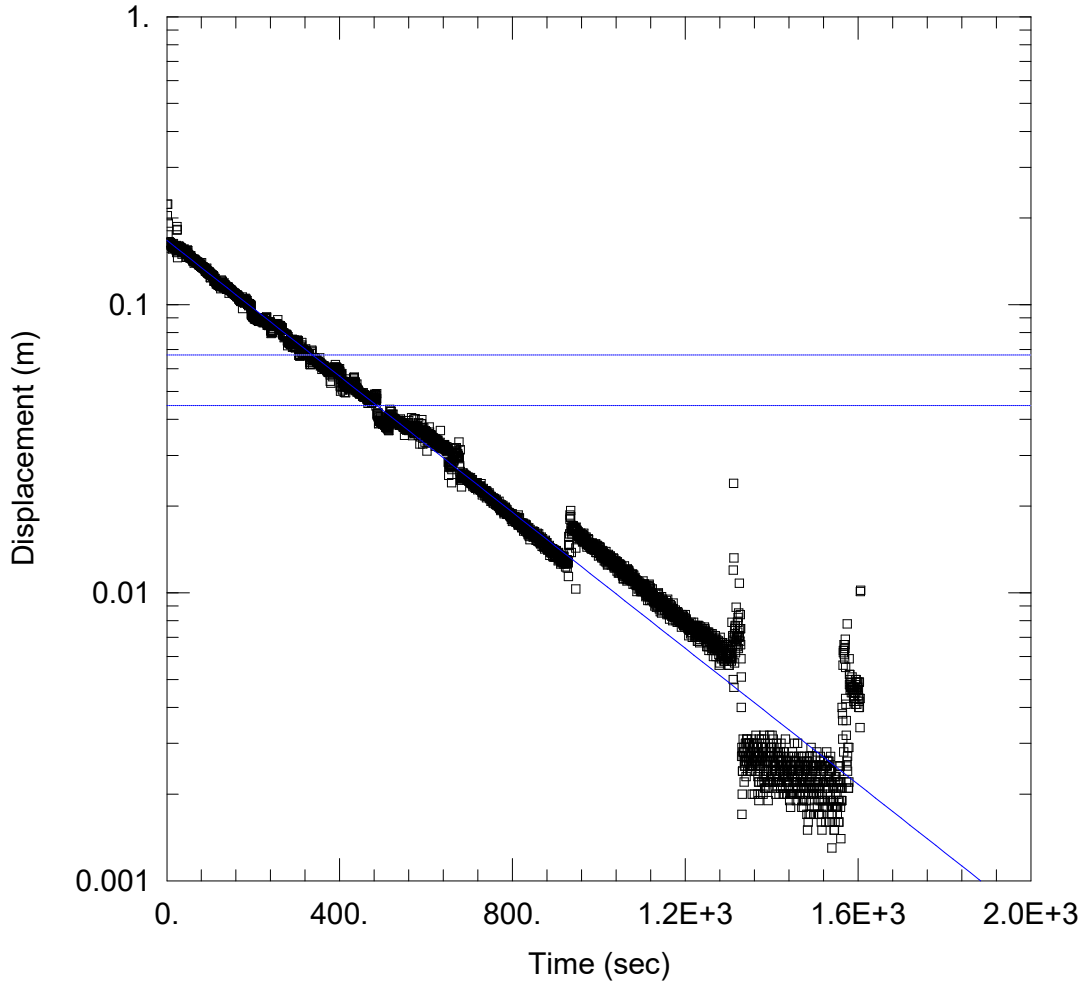
Saturated Thickness: 5.978 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (PZ26-01 RHT)

Initial Displacement: 0.1136 m Static Water Column Height: 2.978 m
Total Well Penetration Depth: 2.978 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 5.534E-8 m/sec y0 = 0.09906 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-02 FHT 1.aqt
Date: 07/22/26 Time: 06:52:57

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

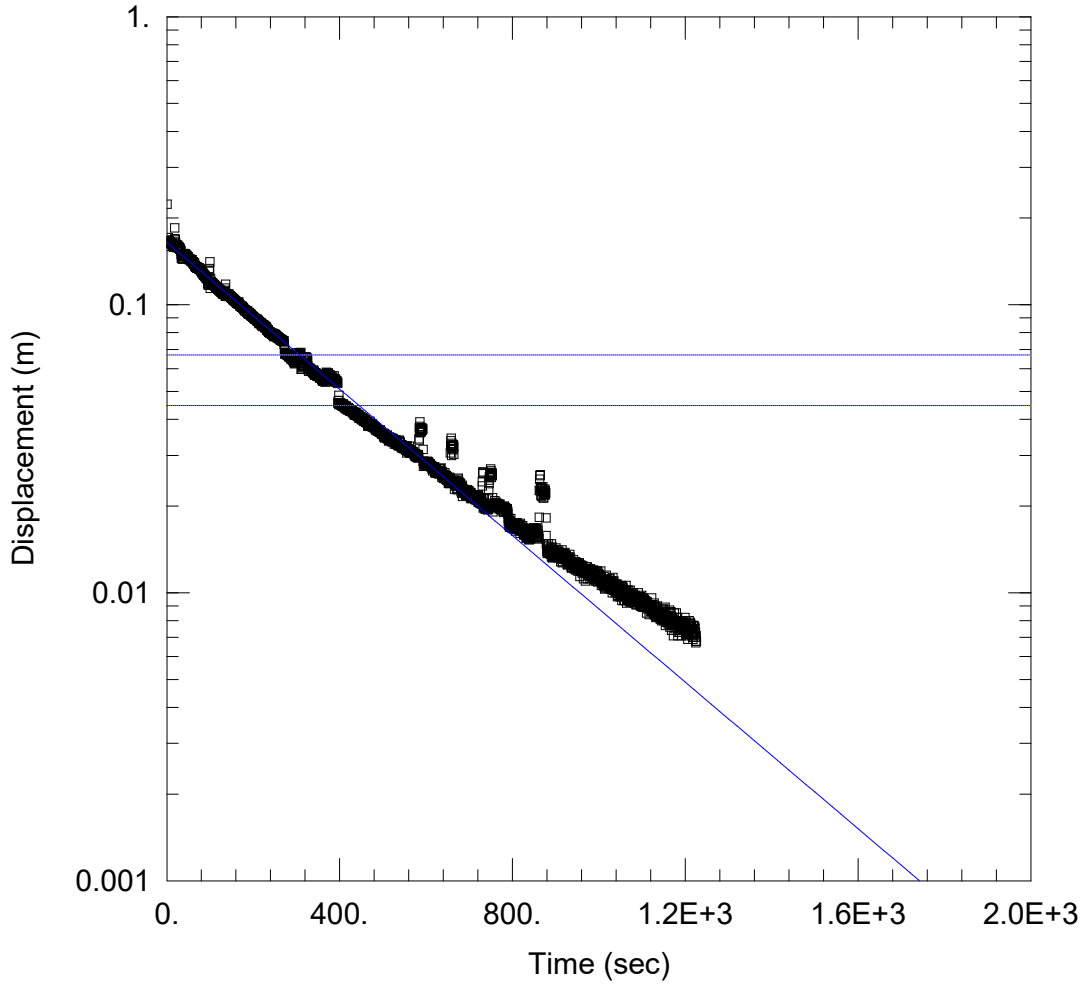
Saturated Thickness: 3.055 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (PZ26-02 FHT 1)

Initial Displacement: 0.2234 m Static Water Column Height: 2.555 m
Total Well Penetration Depth: 2.555 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 9.764E-7 m/sec y0 = 0.1675 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-02 FHT 2.aqt
Date: 07/22/26 Time: 06:55:45

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

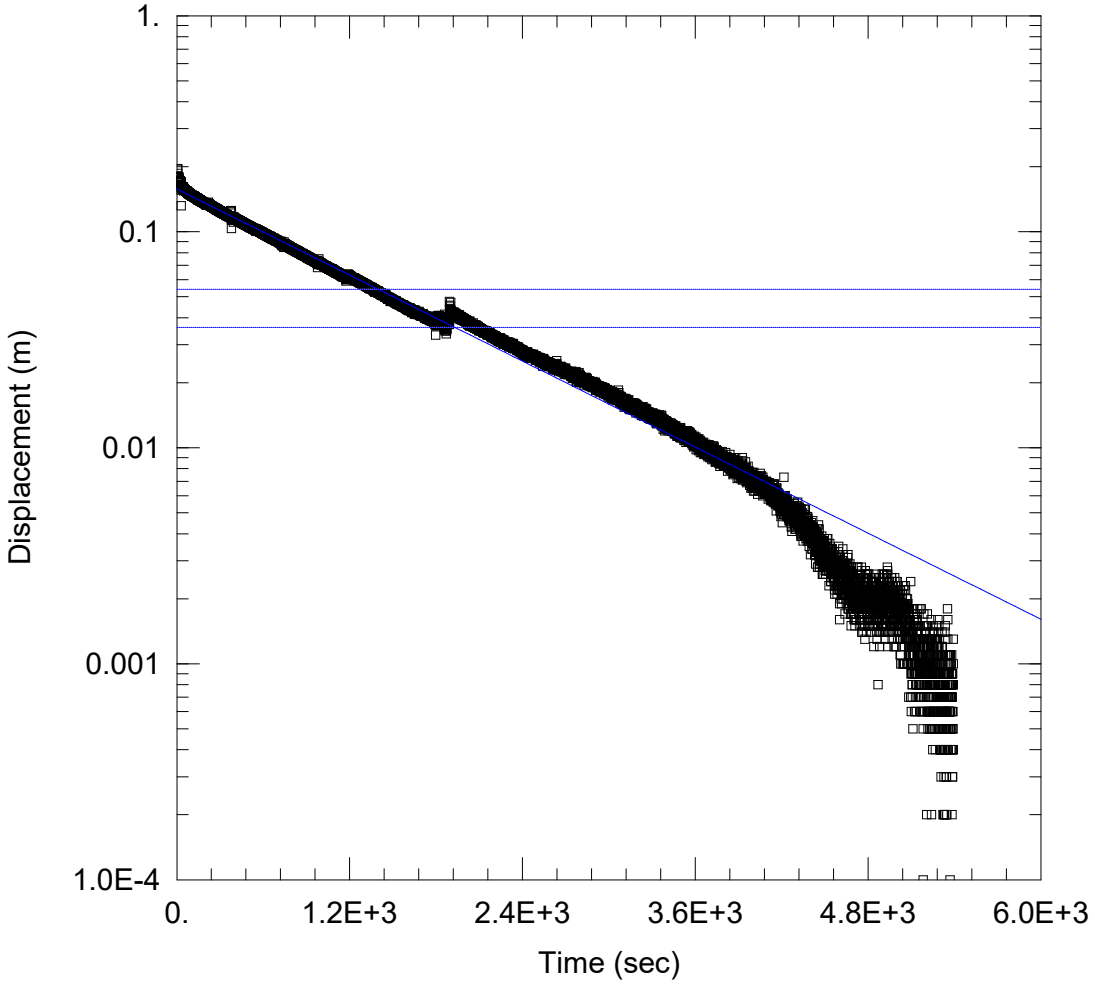
Saturated Thickness: 3.055 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (PZ26-02 FHT 2)

Initial Displacement: 0.2234 m Static Water Column Height: 2.555 m
Total Well Penetration Depth: 2.555 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 1.052E-6 m/sec y0 = 0.1645 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-02 RHT 1.aqt
Date: 07/22/26 Time: 10:56:03

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

Saturated Thickness: 3.055 m Anisotropy Ratio (Kz/Kr): 1.

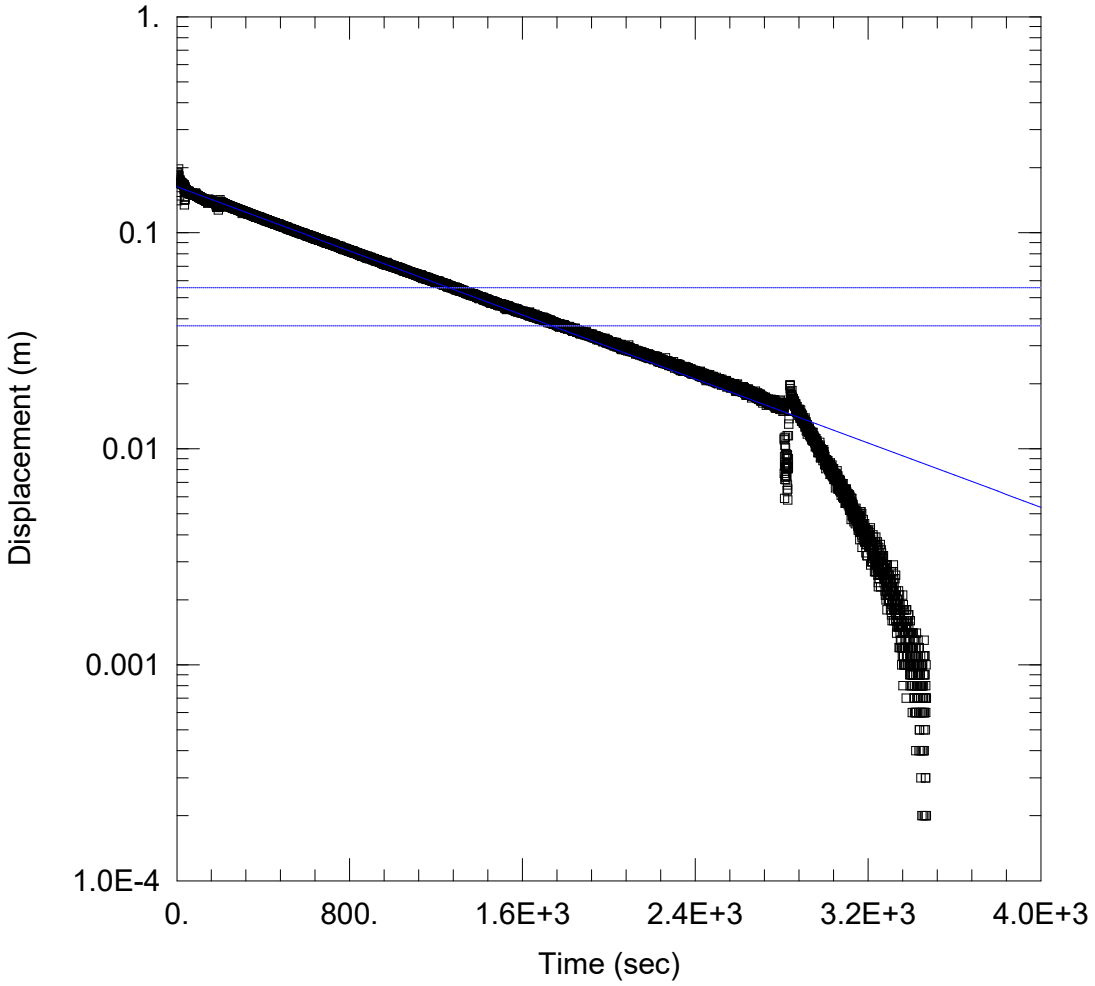
WELL DATA (PZ26-02 RHT 1)

Initial Displacement: 0.1805 m Static Water Column Height: 2.555 m
Total Well Penetration Depth: 2.555 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 2.746E-7 m/sec y0 = 0.1579 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-02 RHT 2.aqt
Date: 07/22/26 Time: 10:55:26

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

Saturated Thickness: 3.055 m Anisotropy Ratio (Kz/Kr): 1.

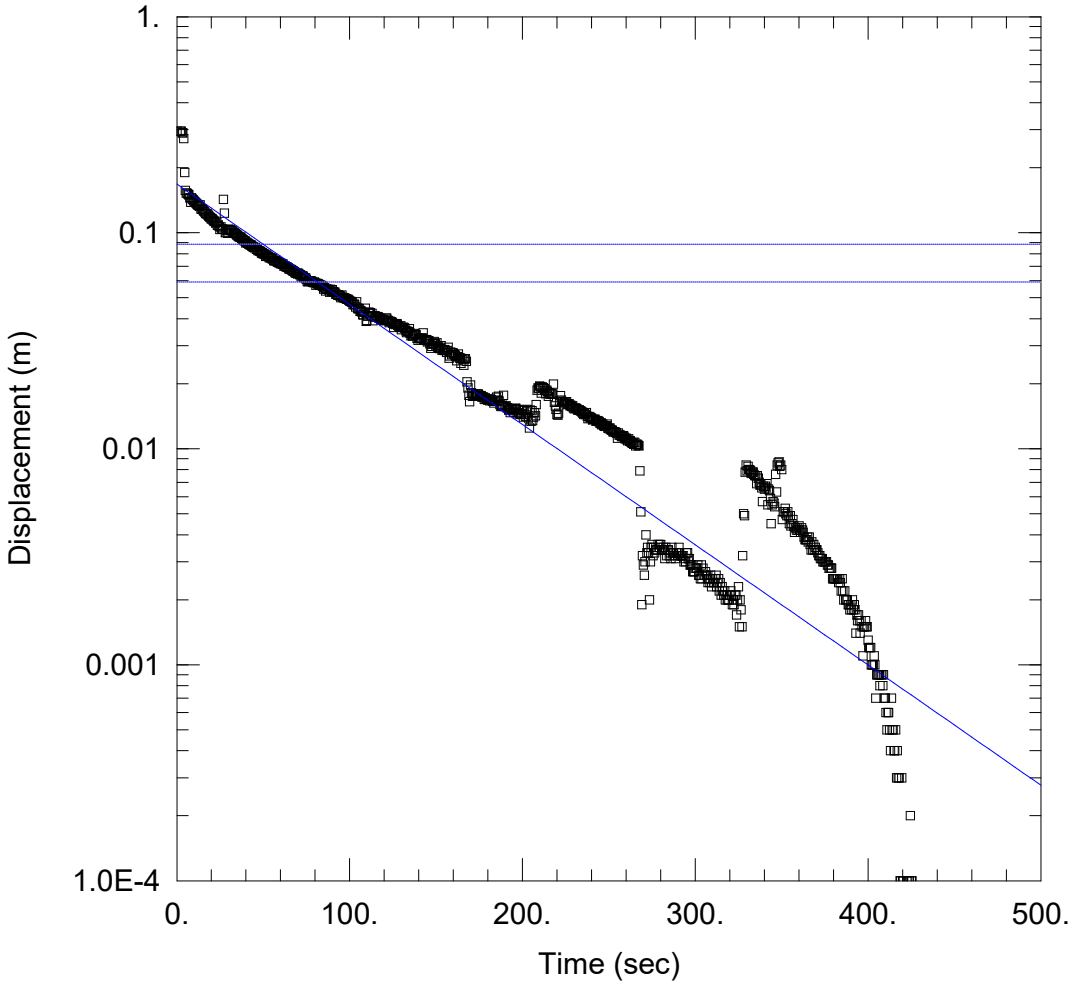
WELL DATA (PZ26-02 RHT 2)

Initial Displacement: 0.1852 m Static Water Column Height: 2.555 m
Total Well Penetration Depth: 2.555 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 3.068E-7 m/sec y0 = 0.1634 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-03 FHT 1.aqt
Date: 07/22/26 Time: 11:43:34

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

Saturated Thickness: 6.948 m Anisotropy Ratio (Kz/Kr): 1.

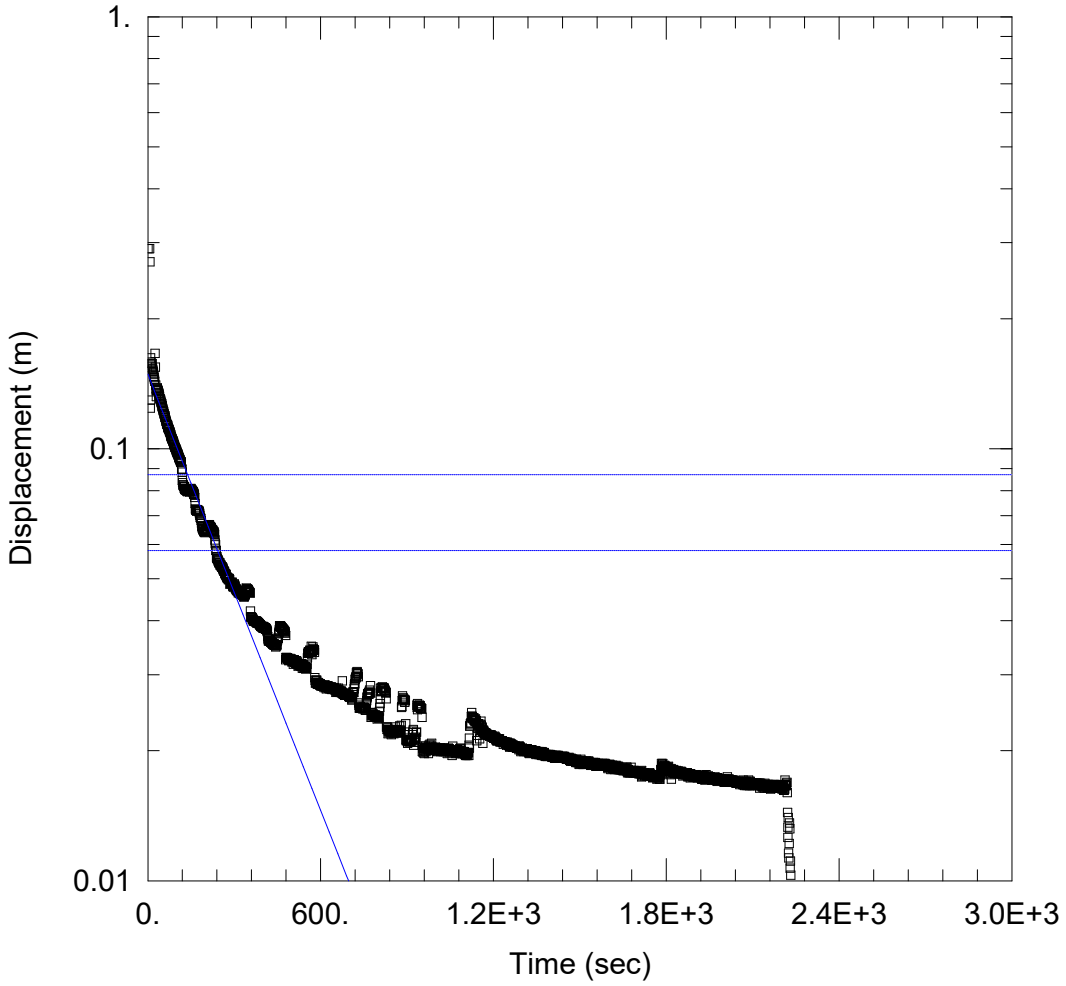
WELL DATA (PZ26-03 FHT 1)

Initial Displacement: 0.2955 m Static Water Column Height: 3.048 m
Total Well Penetration Depth: 3.048 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 4.444E-6 m/sec y0 = 0.168 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-03 FHT 2.aqt
Date: 07/22/26 Time: 11:46:53

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

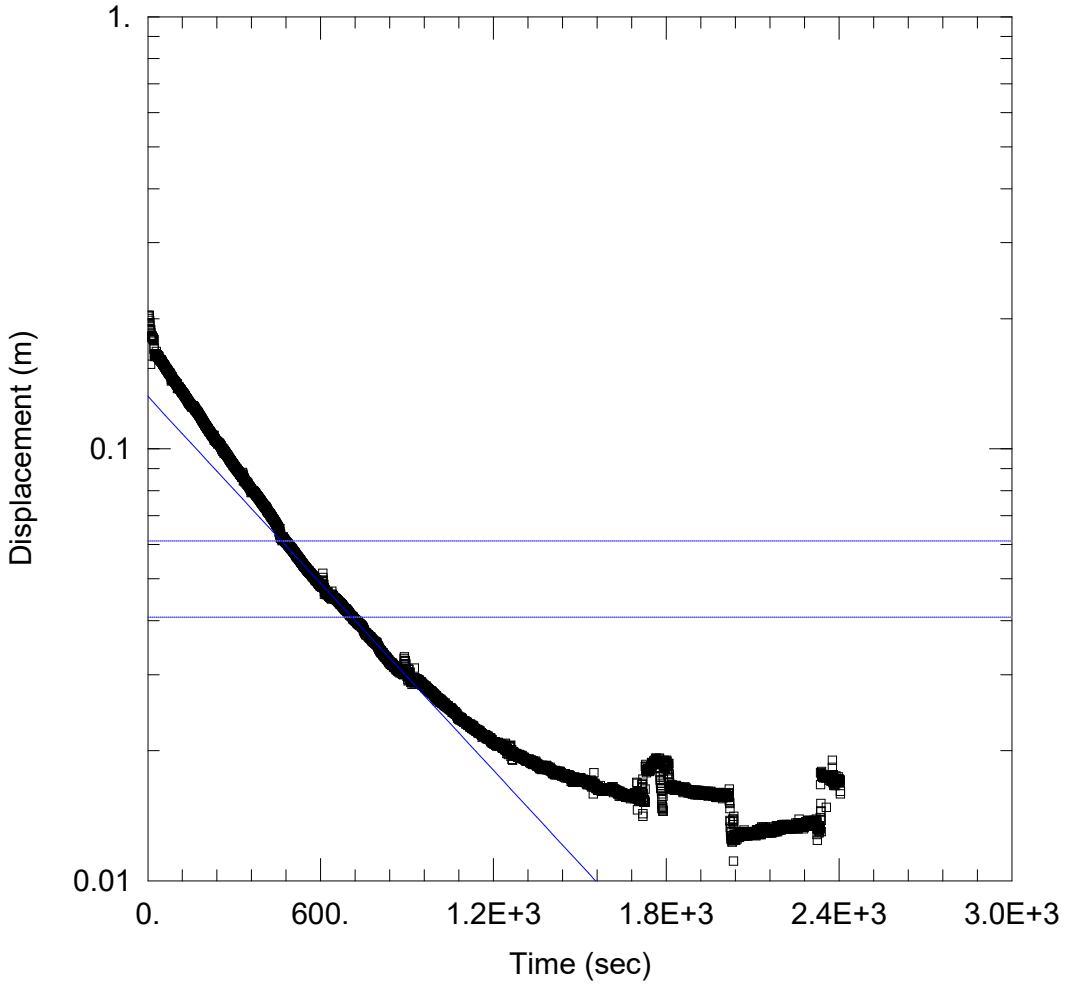
Saturated Thickness: 6.948 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (PZ26-03 FHT 2)

Initial Displacement: 0.2905 m Static Water Column Height: 3.048 m
Total Well Penetration Depth: 3.048 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 1.344E-6 m/sec y0 = 0.1489 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-03 RHT 1.aqt
Date: 07/22/26 Time: 11:38:44

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

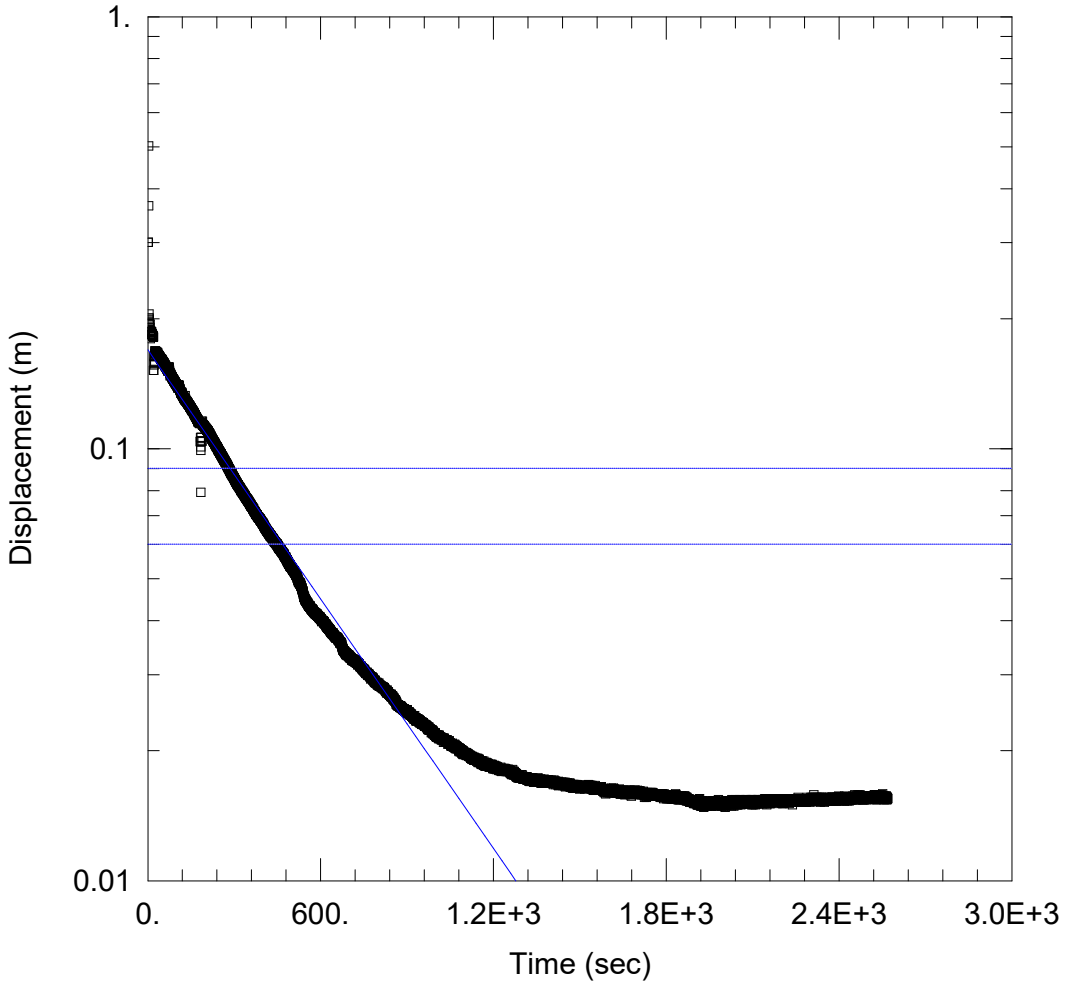
Saturated Thickness: 6.948 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (PZ26-03 RHT 1)

Initial Displacement: 0.2038 m Static Water Column Height: 3.048 m
Total Well Penetration Depth: 3.048 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 5.758E-7 m/sec y0 = 0.1322 m



WELL TEST ANALYSIS

Data Set: V:\...\PZ26-03 RHT 2.aqt
Date: 07/22/26 Time: 11:40:55

PROJECT INFORMATION

Company: WGA
Project: 260929
Location: 2025 Ohaupo Rd
Test Date: 20/07/26

AQUIFER DATA

Saturated Thickness: 6.948 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (PZ26-03 RHT 2)

Initial Displacement: 0.3005 m Static Water Column Height: 3.048 m
Total Well Penetration Depth: 3.048 m Screen Length: 1.5 m
Casing Radius: 0.02 m Well Radius: 0.04 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 7.678E-7 m/sec y0 = 0.1695 m

APPENDIX D

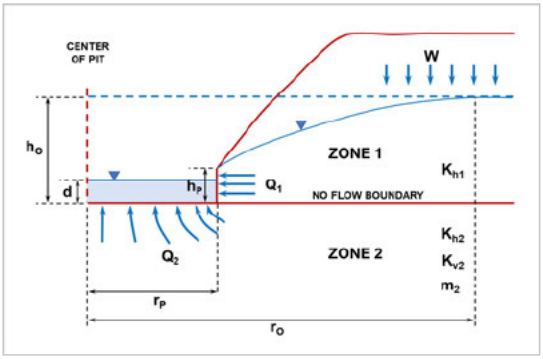
DRAWDOWN CALCULATIONS FOR STORMWATER WETLANDS



INFLOW TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	Wetland A - Construction Dry
Date:	Tuesday, 11 August 2026
Document #:	WGA260929-RP-HG-0001



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

C $$Q_2 = 4r_p\left(\frac{K_{h2}}{m_2}\right)\left(h_0 - d\right),$$

D $$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1E-08	**m/s**	recharge flux
K_{h1}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.40E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	19.95	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	40	m
Known h_O	1	m
h_O calculated using Equation A	2.2	m
Q_1	4.8E-05	m^3/s
	4.1E+00	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	3.5E-05	m^3/s
	3.1E+00	m^3/day

Total Pit Inflow	**Q_{TOT}**	8.3E-05	m^3/s
		7.18	m^3/day

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

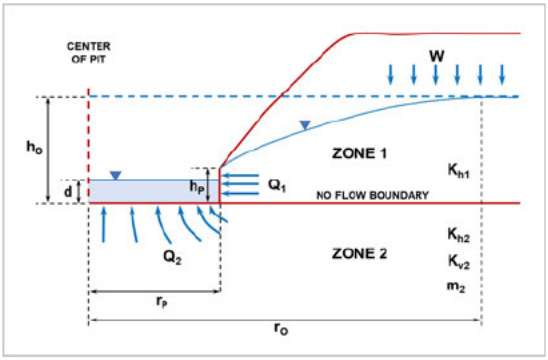
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	Wetland A - Construction Dry
Date:	Tuesday, 11 August 2026
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$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

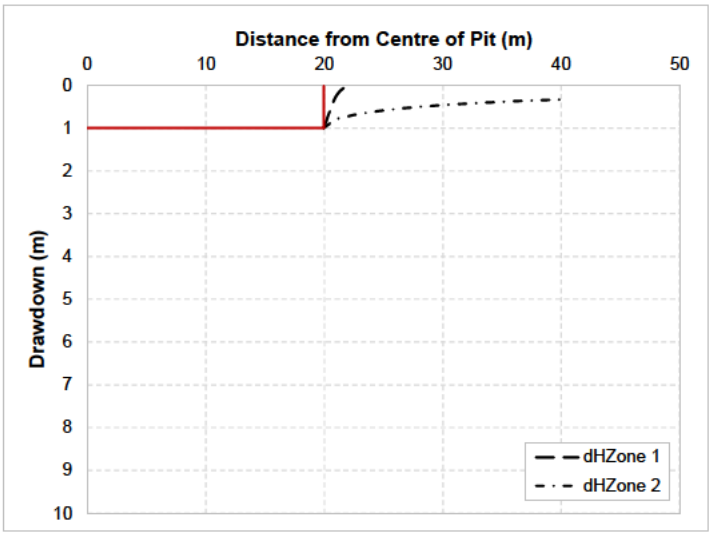
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	19.95	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 40 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
19.95	1	1.0
21.0	0.3	0.8
22.0	0.0	0.7
23.0	-0.2	0.7
24.0	-0.4	0.6
25.0	-0.5	0.6
26.0	-0.6	0.6
27.0	-0.7	0.5
28.0	-0.8	0.5
29.0	-0.8	0.5
30.0	-0.9	0.5
31.0	-1.0	0.4
32.0	-1.0	0.4
33.0	-1.04	0.4
34.0	-1.07	0.4
35.0	-1.09	0.4
36.0	-1.12	0.4
37.0	-1.13	0.4
38.0	-1.14	0.4
39.0	-1.15	0.3
40.0	-1.15	0.3

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

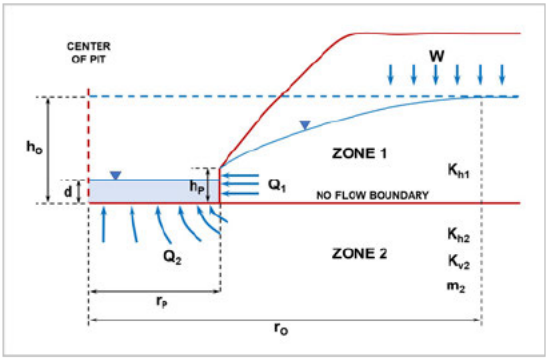
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT: Te Awamutu Developments Ltd
PROJECT: Te Awamutu Development
Project #: WGA260929
Location: Wetland A - Construction Dry
Date: Tuesday, 11 August 2026
Document #: WGA260929-RP-HG-0001



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

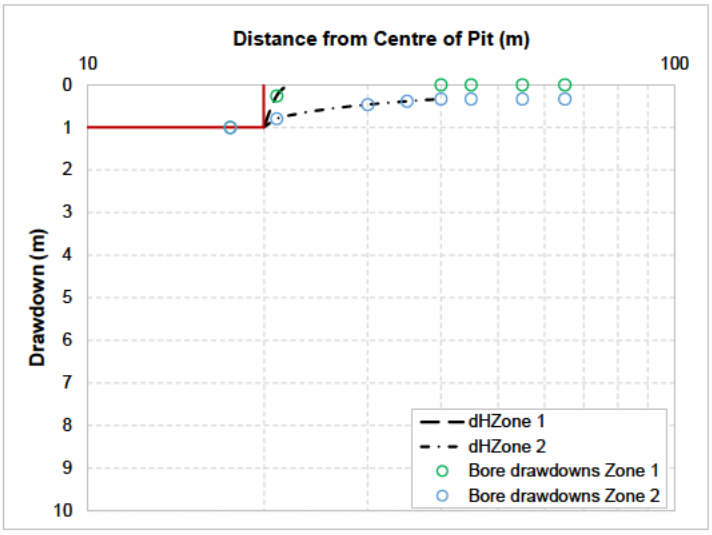
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	19.95	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 40 m
Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	1.0	1.0
21.0	0.3	0.8
30.0	-0.9	0.5
35.0	-1.1	0.4
40.0	0.0	0.3
45.0	0.0	0.3
55.0	0.0	0.3
65.0	0.0	0.3

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

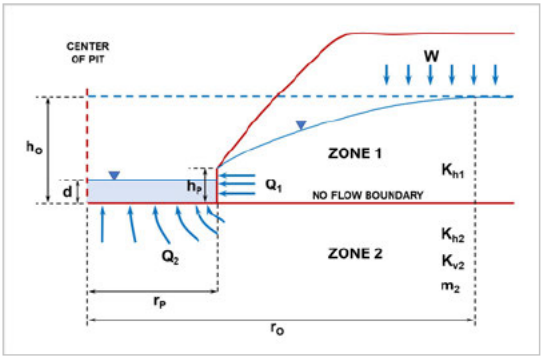
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

INFLOW TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	Wetland B - Construction Dry
Date:	Tuesday, 15 September 2026
Document #:	WGA260929-RP-HG-0001



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

C $$Q_2 = 4r_p\left(\frac{K_{h2}}{m_2}\right)\left(h_0 - d\right),$$

D $$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1E-08	**m/s**	recharge flux
K_{h1}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.40E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1.5	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	26.87	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	41	m
Known h_O	1.5	m
h_O calculated using Equation A	1.5	m
Q_1	4.0E-05	m^3/s
	3.4E+00	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	7.1E-05	m^3/s
	6.2E+00	m^3/day

Total Pit Inflow **Q_{TOT}**	1.1E-04	m^3/s
	9.61	m^3/day

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

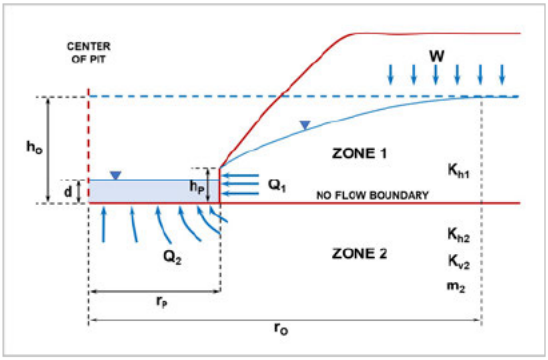
Analysed by: EC 15/09/2026
Checked by: CHO 15/09/2026

Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

WGA

CLIENT: Te Awamutu Developments Ltd
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$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

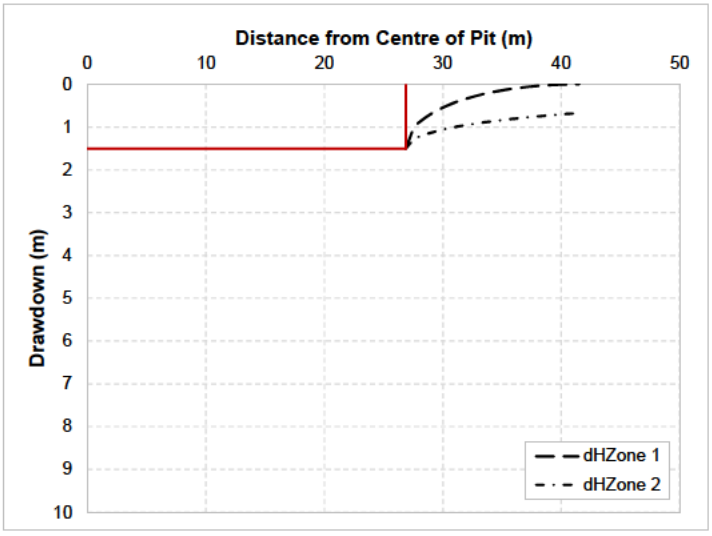
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1.5	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	26.87	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 41 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
26.87	1.5	1.5
27.6	1.0	1.3
28.3	0.8	1.2
29.1	0.7	1.1
29.8	0.6	1.1
30.5	0.5	1.0
31.3	0.4	1.0
32.0	0.3	1.0
32.7	0.3	0.9
33.5	0.2	0.9
34.2	0.2	0.9
34.9	0.1	0.8
35.6	0.1	0.8
36.4	0.09	0.8
37.1	0.06	0.8
37.8	0.04	0.8
38.6	0.03	0.7
39.3	0.01	0.7
40.0	0.01	0.7
40.8	0.00	0.7
41.5	0.00	0.7

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

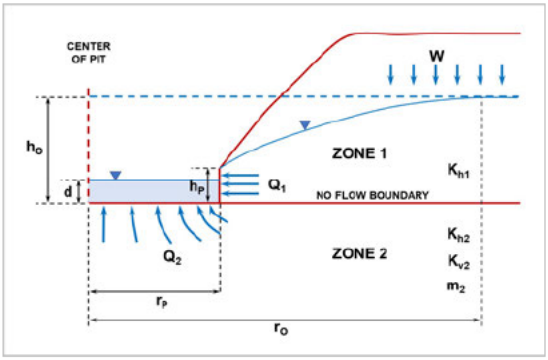
Analysed by: EC 15/09/2026
Checked by: CHO 15/09/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT: Te Awamutu Developments Ltd
PROJECT: Te Awamutu Development
Project #: WGA260929
Location: Wetland B - Construction Dry
Date: Tuesday, 15 September 2026
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$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

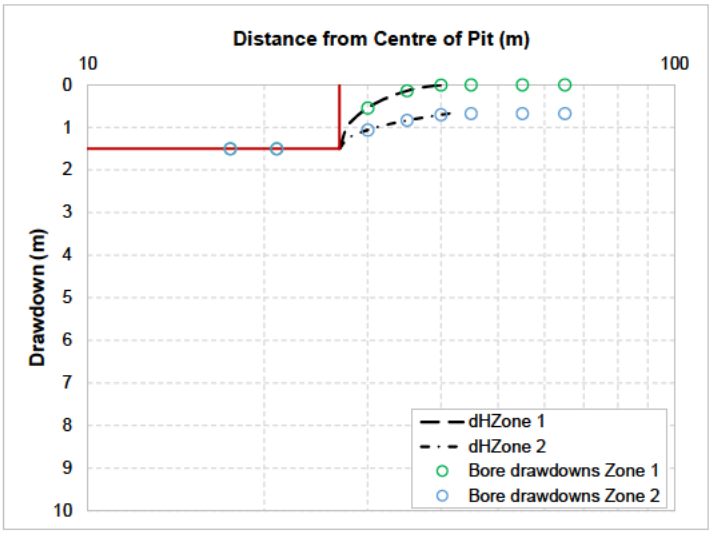
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1.5	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	26.87	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 41 m
Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	1.5	1.5
21.0	1.5	1.5
30.0	0.5	1.1
35.0	0.1	0.8
40.0	0.0	0.7
45.0	0.0	0.7
55.0	0.0	0.7
65.0	0.0	0.7

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

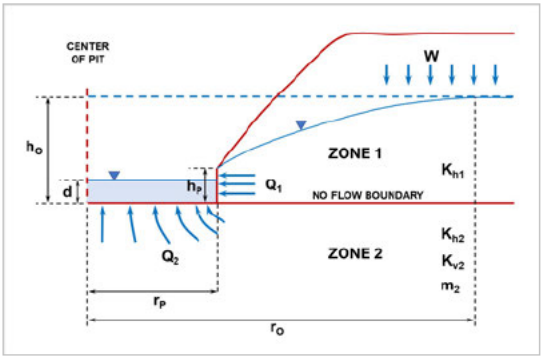
Analysed by: EC 15/09/2026
Checked by: CHO 15/09/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

INFLOW TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	Wetland C - Construction Dry
Date:	Tuesday, 11 August 2026
Document #:	WGA260929-RP-HG-0001



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

C $$Q_2 = 4r_p\left(\frac{K_{h2}}{m_2}\right)\left(h_0 - d\right),$$

D $$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1E-08	**m/s**	recharge flux
K_{h1}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.40E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	17.82	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	49	m
Known h_O	1	m
h_O calculated using Equation A	3.5	m
Q_1	8.2E-05	m^3/s
	7.1E+00	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	3.2E-05	m^3/s
	2.7E+00	m^3/day

Total Pit Inflow	**Q_{TOT}**	1.1E-04	m^3/s
		9.78	m^3/day

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

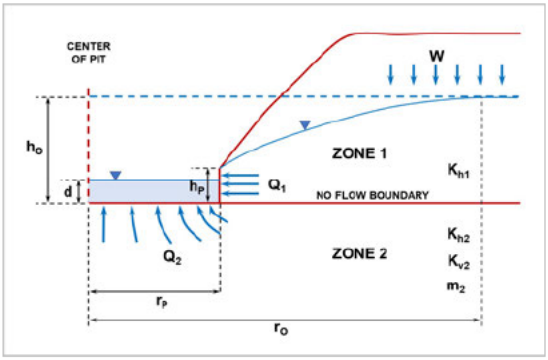
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Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

WGA

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PROJECT:	Te Awamutu Development
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$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

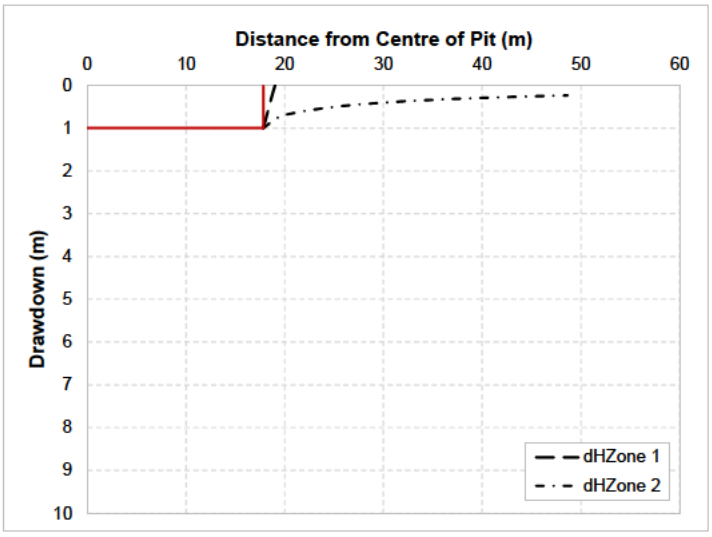
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	17.82	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 49 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.82	1	1.0
19.4	-0.2	0.7
20.9	-0.7	0.7
22.4	-1.0	0.6
24.0	-1.3	0.5
25.5	-1.5	0.5
27.1	-1.7	0.5
28.6	-1.8	0.4
30.2	-1.9	0.4
31.7	-2.0	0.4
33.2	-2.1	0.4
34.8	-2.2	0.3
36.3	-2.3	0.3
37.9	-2.33	0.3
39.4	-2.38	0.3
40.9	-2.42	0.3
42.5	-2.45	0.3
44.0	-2.47	0.3
45.6	-2.49	0.3
47.1	-2.50	0.2
48.6	-2.50	0.2

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

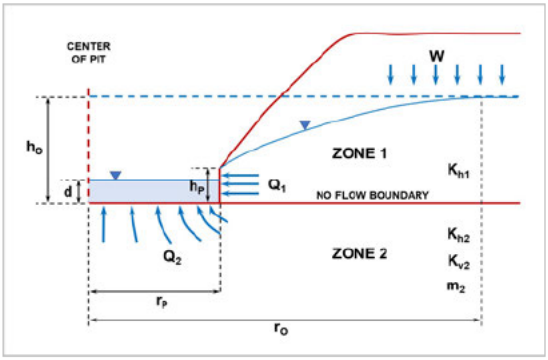
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	Wetland C - Construction Dry
Date:	Tuesday, 11 August 2026
Document #:	WGA260929-RP-HG-0001



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

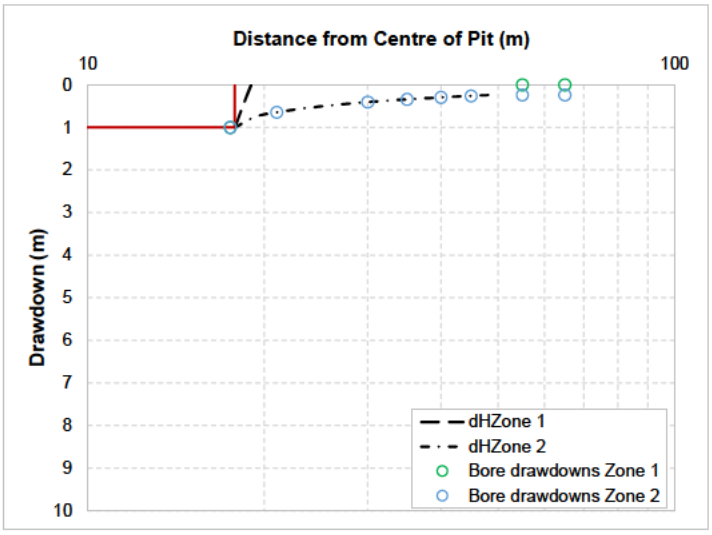
$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	17.82	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 49 m

Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	1.0	1.0
21.0	-0.7	0.6
30.0	-1.9	0.4
35.0	-2.2	0.3
40.0	-2.4	0.3
45.0	-2.5	0.3
55.0	0.0	0.2
65.0	0.0	0.2

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

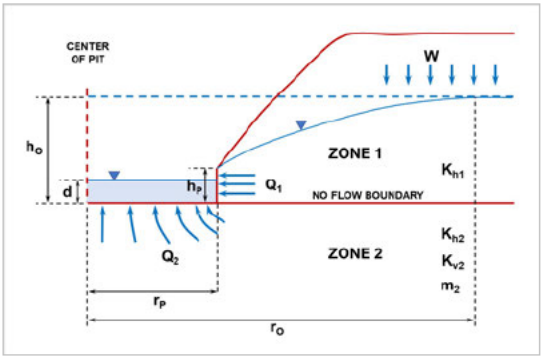
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

INFLOW TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	Wetland D - Construction Dry
Date:	Tuesday, 11 August 2026
Document #:	WGA260929-RP-HG-0001



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

C $$Q_2 = 4r_p\left(\frac{K_{h2}}{m_2}\right)\left(h_0 - d\right),$$

D $$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1E-08	**m/s**	recharge flux
K_{h1}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.40E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	17.64	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	28	m
Known h_O	1	m
h_O calculated using Equation A	1.1	m
Q_1	2.0E-05	m^3/s
	1.7E+00	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	3.1E-05	m^3/s
	2.7E+00	m^3/day

Total Pit Inflow	**Q_{TOT}**	5.1E-05	m^3/s
		4.38	m^3/day

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

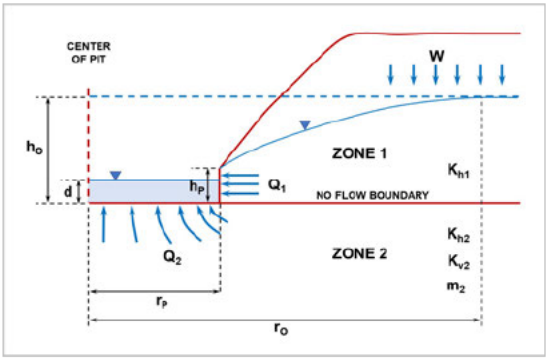
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

WGA

CLIENT: Te Awamutu Developments Ltd
PROJECT: Te Awamutu Development
Project #: WGA260929
Location: Wetland D - Construction Dry
Date: Tuesday, 11 August 2026
Document #: WGA260929-RP-HG-0001



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

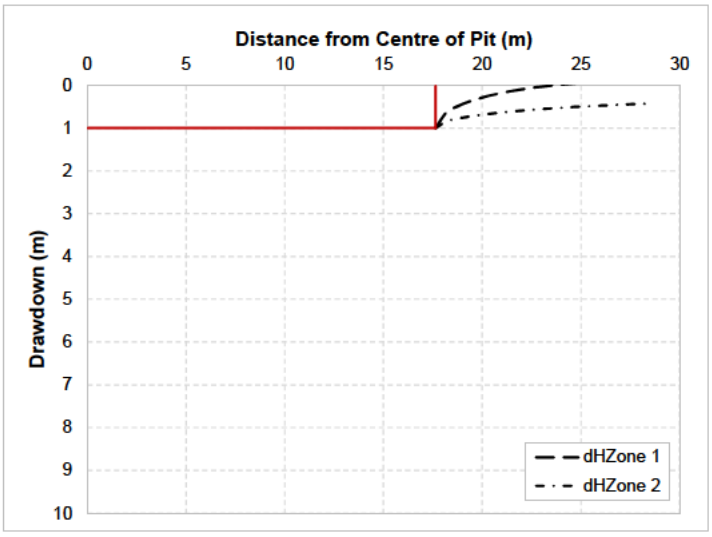
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	17.64	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 28 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.64	1	1.0
18.2	0.6	0.8
18.7	0.5	0.8
19.2	0.4	0.7
19.8	0.3	0.7
20.3	0.3	0.7
20.8	0.2	0.6
21.4	0.1	0.6
21.9	0.1	0.6
22.4	0.1	0.6
23.0	0.0	0.6
23.5	0.0	0.5
24.0	0.0	0.5
24.6	-0.04	0.5
25.1	-0.06	0.5
25.6	-0.07	0.5
26.2	-0.08	0.5
26.7	-0.09	0.5
27.2	-0.10	0.4
27.8	-0.10	0.4
28.3	-0.10	0.4

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

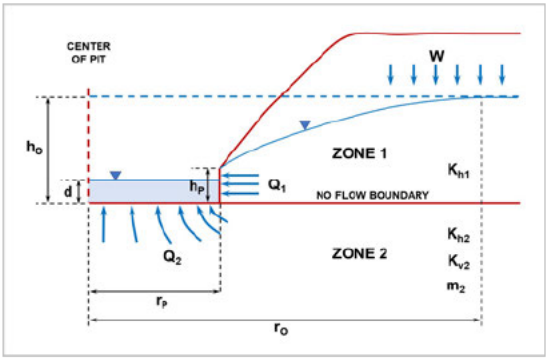
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT: Te Awamutu Developments Ltd
PROJECT: Te Awamutu Development
Project #: WGA260929
Location: Wetland D - Construction Dry
Date: Tuesday, 11 August 2026
Document #: WGA260929-RP-HG-0001



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

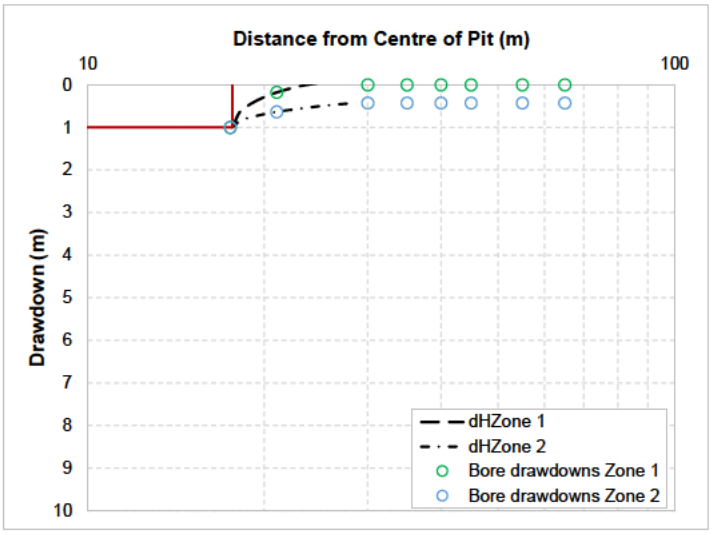
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	17.64	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 28 m
Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	1.0	1.0
21.0	0.2	0.6
30.0	0.0	0.4
35.0	0.0	0.4
40.0	0.0	0.4
45.0	0.0	0.4
55.0	0.0	0.4
65.0	0.0	0.4

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

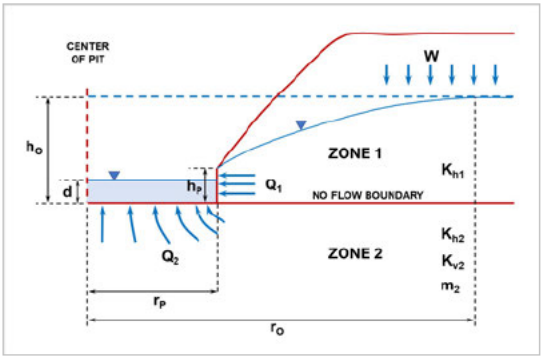
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

INFLOW TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	Wetland E - Construction Dry
Date:	Tuesday, 11 August 2026
Document #:	WGA260929-RP-HG-0001



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

C $$Q_2 = 4r_p\left(\frac{K_{h2}}{m_2}\right)\left(h_0 - d\right),$$

D $$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1E-08	**m/s**	recharge flux
K_{h1}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.40E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	18.68	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	51	m
Known h_O	1	m
h_O calculated using Equation A	3.7	m
Q_1	9.1E-05	m^3/s
	7.8E+00	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	3.3E-05	m^3/s
	2.9E+00	m^3/day

Total Pit Inflow

Q_{TOT}	1.2E-04	m^3/s
	10.70	m^3/day

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

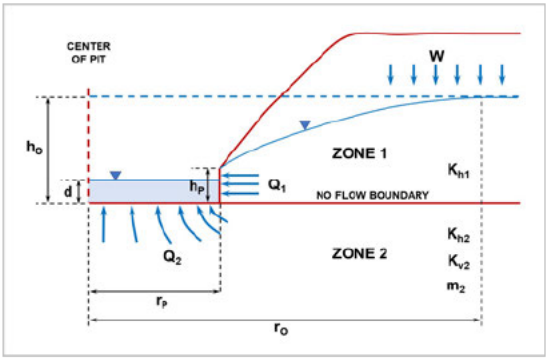
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

WGA

CLIENT: Te Awamutu Developments Ltd
PROJECT: Te Awamutu Development
Project #: WGA260929
Location: Wetland E - Construction Dry
Date: Tuesday, 11 August 2026
Document #: WGA260929-RP-HG-0001



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{\left(r^2 - r_p^2\right)}{2}\right]}$$

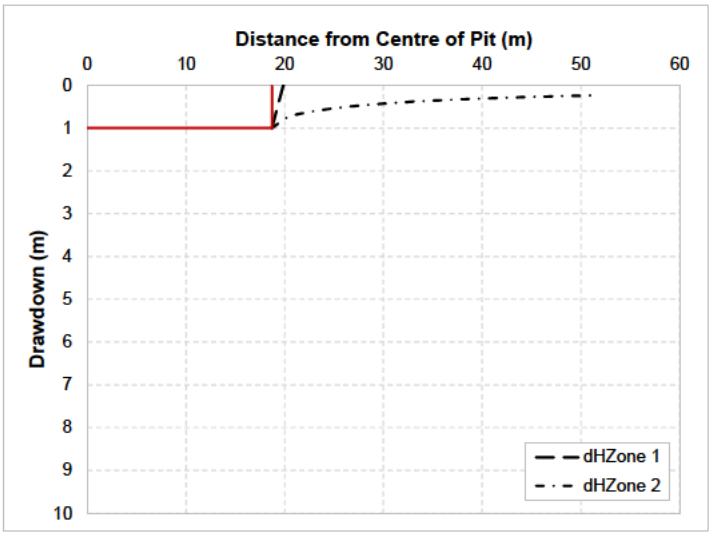
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	18.68	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 51 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
18.68	1	1.0
20.3	-0.3	0.7
21.9	-0.8	0.6
23.6	-1.1	0.6
25.2	-1.4	0.5
26.8	-1.6	0.5
28.4	-1.8	0.5
30.1	-2.0	0.4
31.7	-2.1	0.4
33.3	-2.2	0.4
35.0	-2.3	0.4
36.6	-2.4	0.3
38.2	-2.5	0.3
39.8	-2.52	0.3
41.5	-2.57	0.3
43.1	-2.61	0.3
44.7	-2.65	0.3
46.4	-2.67	0.3
48.0	-2.69	0.3
49.6	-2.70	0.2
51.2	-2.70	0.2

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

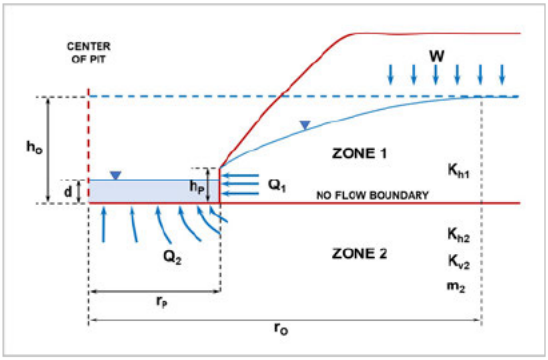
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT: Te Awamutu Developments Ltd
PROJECT: Te Awamutu Development
Project #: WGA260929
Location: Wetland E - Construction Dry
Date: Tuesday, 11 August 2026
Document #: WGA260929-RP-HG-0001



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

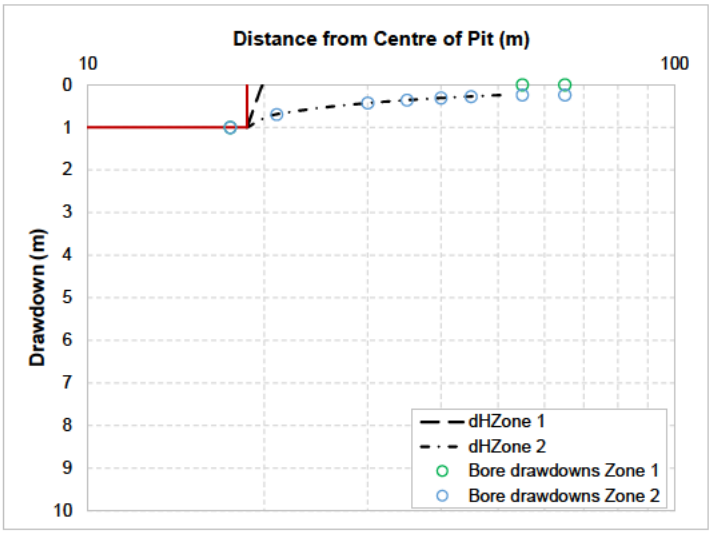
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	18.68	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 51 m
Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	1.0	1.0
21.0	-0.5	0.7
30.0	-2.0	0.4
35.0	-2.3	0.4
40.0	-2.5	0.3
45.0	-2.6	0.3
55.0	0.0	0.2
65.0	0.0	0.2

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

APPENDIX E

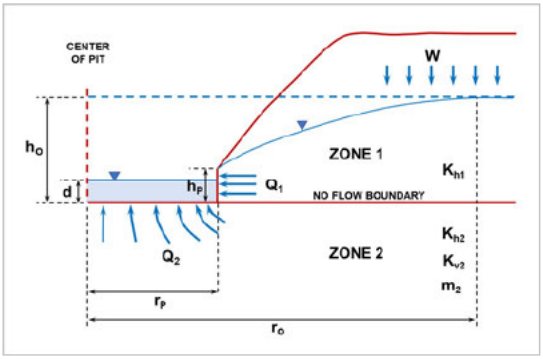
DRAWDOWN CALCULATIONS FOR WASTEWATER INFRASTRUCTURE



INFLOW TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	WWPS B - Construction
Date:	Tuesday, 11 August 2026
Document #:	WGA260929-RP-HG-0001



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

C $$Q_2 = 4r_p\left(\frac{K_{h2}}{m_2}\right)\left(h_0 - d\right),$$

D $$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1E-08	**m/s**	recharge flux
K_{h1}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.40E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.40E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1.5	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	8	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	21	m
Known h_O	1.5	m
h_O calculated using Equation A	1.5	m
Q_1	1.5E-05	m^3/s
	1.3E+00	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	2.1E-05	m^3/s
	1.8E+00	m^3/day

Total Pit Inflow **Q_{TOT}**	3.7E-05	m^3/s
	3.17	m^3/day

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

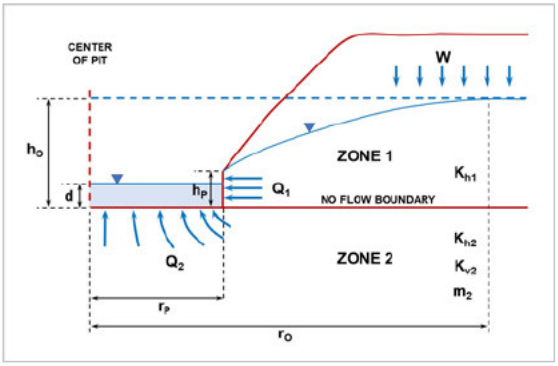
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	WWPS B - Construction
Date:	Tuesday, 11 August 2026
Document #:	WGA260929-RP-HG-0001



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{\left(r^2 - r_p^2\right)}{2}\right]}$$

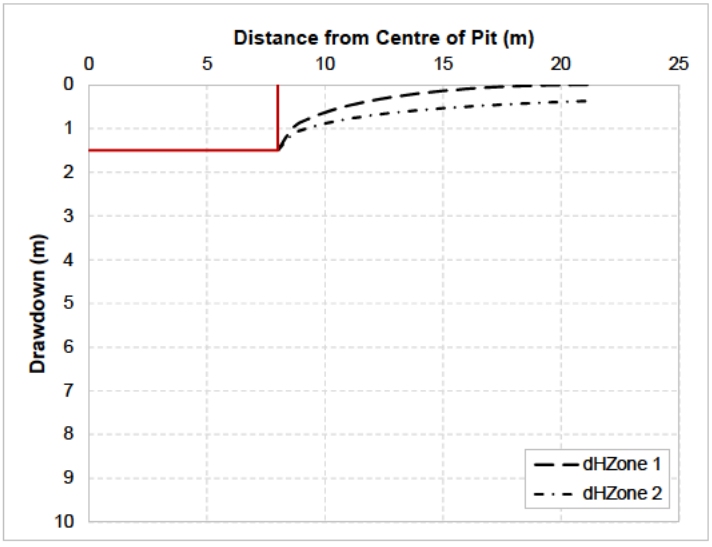
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1.5	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	8	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 21 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
8	1.5	1.5
8.7	1.0	1.1
9.3	0.8	1.0
10.0	0.6	0.9
10.7	0.5	0.8
11.3	0.4	0.7
12.0	0.4	0.7
12.6	0.3	0.7
13.3	0.2	0.6
14.0	0.2	0.6
14.6	0.2	0.6
15.3	0.1	0.5
16.0	0.1	0.5
16.6	0.07	0.5
17.3	0.05	0.5
18.0	0.04	0.4
18.6	0.02	0.4
19.3	0.01	0.4
19.9	0.01	0.4
20.6	0.00	0.4
21.3	0.00	0.4

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

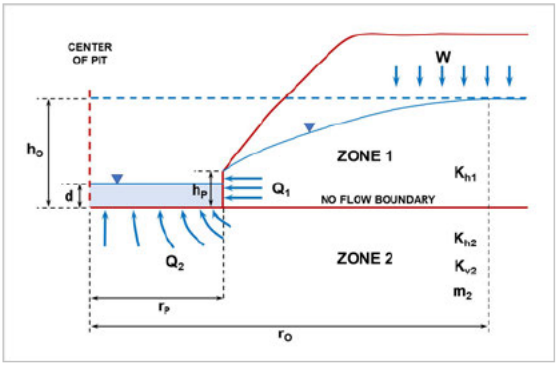
Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Te Awamutu Developments Ltd
PROJECT:	Te Awamutu Development
Project #:	WGA260929
Location:	WWPS B - Construction
Date:	Tuesday, 11 August 2026
Document #:	WGA260929-RP-HG-0001



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{\left(r^2 - r_p^2\right)}{2}\right]}$$

$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

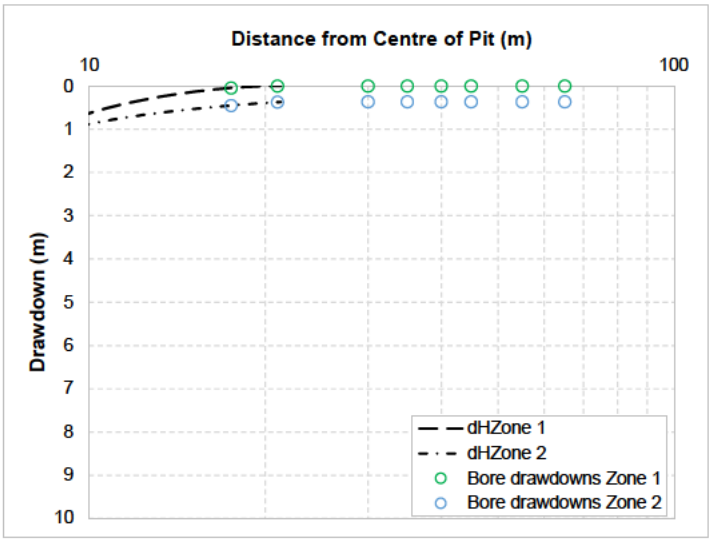
$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

W	1.27E-08	**m/s**	recharge flux
K_{h1}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	1.4E-06	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	1.4E-07	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1.5	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	8	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 21 m

Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	0.0	0.5
21.0	0.0	0.4
30.0	0.0	0.4
35.0	0.0	0.4
40.0	0.0	0.4
45.0	0.0	0.4
55.0	0.0	0.4
65.0	0.0	0.4

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

Analysed by: EC 10/08/2026
Checked by: CHO 17/08/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

WGA

An AtkinsRéalis Company

FOR FURTHER INFORMATION CONTACT:

Clare Houlbrooke
Principal Hydrogeologist

M [redacted]
E [redacted]

WGA.COM.AU
WGANZ.CO.NZ